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Saudi Building Code National Committee

كود البناء السعودي
Saudi Building Code

Saudi Masonry Code

SBC 305 E

Requirements

كود المنشآت الطوبية

2018

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Saudi Building Code for Masonry Structures
SBC 305

Key List of the Saudi Codes: Designations and brief titles			
Title	**Code Req.**[1]	**Code & Com.**[2]	**Arabic Prov.**[3]
The General Building Code	SBC 201-CR	SBC 201-CC	SBC 201-AR
Structural – Loading and Forces	SBC 301-CR	SBC 301-CC	SBC 301-AR
Structural – Construction	SBC 302- CR		SBC 302-AR
Structural – Soil and Foundations	SBC 303- CR	SBC 303-CC	SBC 303-AR
Structural – Concrete Structures	SBC 304- CR	SBC 304-CC	SBC 304-AR
Structural – Masonry Structures	SBC 305-CR	SBC 305-CC	SBC 305-AR
Structural – Steel Structures			
Electrical Code	SBC 401-CR		SBC 401-AR
Mechanical Code	SBC 501- CR	SBC 501-CC	SBC 501-AR
Energy Conservation-Nonresidential		SBC 601- CC	SBC 601- AR
Energy Conservation-Residential		SBC 602- CC	SBC 602- AR
Plumbing Code	SBC 701- CR	SBC 701-CC	SBC 701-AR
Private sewage Code	SBC 702- CR		SBC 702-AR
Fire Code	SBC 801- CR	SBC 801-CC	SBC 801-AR
Existing Buildings Code	SBC 901- CR	SBC 901-CC	SBC 901-AR
Green Construction Code	SBC 1001- CR	SBC 1001-CC	SBC 1001-AR
Residential Building Code*	SBC 1101- CR	SBC 1101-CC	SBC 1101-AR
Fuel Gas Code*	SBC 1201- CR	SBC 1201-CC	SBC 1201-AR

1. *CR: Code Requirements without Commentary*
2. *CC: Code Requirements with Commentary*
3. *AR: Arabic Code Provisions*

* *Under Development*

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PREFACE

The Saudi Building Code for Masonry Structures (SBC 305) provides minimum requirements for the structural design and construction of masonry elements consisting of masonry units bedded in mortar. The first edition of SBC 305 was published in the year of 2007. SBC 305-18 is the second edition SBC 305 and addresses the structural design of both structural and non-structural masonry elements. The nonstructural elements are primarily masonry veneer, glass unit masonry, and masonry partitions. Structural design aspects of non-structural masonry elements include, but are not limited to, gravity and lateral support, and load transfer to supporting elements. The requirements provided in SBC 305 are related to contract documents; quality assurance; materials; placement of embedded items; analysis and design; strength and serviceability; flexural and axial loads; shear; details and development of reinforcement; walls; columns; pilasters; beams and lintels; seismic design requirements; glass unit masonry; and veneers. An empirical design method applicable to buildings meeting specific location and construction criteria is also included in this Code.

The current edition of the Code has been substantially reorganized and reformatted relative to its 2007 edition. The code is reorganized into five parts, 14 chapters and six appendices. The reorganization was in response to past requests concerning the difficulty in finding provisions. The new layout is more user-friendly and will better facilitate the use of the design provisions.

ACI 530/530.1-13 is the base code in the development of this Code. Saudi Building Code National Committee (SBCNC) has made an agreement with the ACI to use their materials and modify them as per the local construction needs and regulatory requirements of Saudi Arabia. ACI is not responsible for any modifications or changes the SBCNC has made to accommodate local conditions.

The writing process of SBC 305-18 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made in its base code (ACI 530/530.1-13) to meet the local weather, materials, construction and regulatory requirements.

The committees responsible for SBC 305 Code have taken all precautions to avoid ambiguities, omissions, and errors in the document. Despite these efforts, the users of SBC 305 may find information or requirements that may be subject to more than one interpretation or may be incomplete. The SBCNC alone possesses the authority and responsibility for updating, modifying and interpreting the Code.

It is a common assumption that engineering knowledge is a prerequisite in understanding code provisions and requirements; thus, the code is oriented towards individuals who possess the background knowledge to evaluate the significance and limitations of its content and recommendations. They shall be able to determine the applicability of all regulatory limitations before applying the Code and must comply with all applicable laws and regulations.

The requirements related to administration and enforcement of this Code are advisory only. SBCNC and governmental organizations, in charge of enforcing this Code, possess the authority to modify these administrative requirements.

SUMMARY OF CHAPTERS

The entire SBC 305-18 is divided into 14 chapters and six appendices. A brief outline of these chapters and appendices is given below:

Chapter 1. General Requirements—This chapter includes a number of provisions that explain where SBC 305 Code applies and how it is to be interpreted. It also lists some of the important items of information that must be included in the project drawings or project specifications. All the Standards, or specific sections thereof, cited in this Code, including Appendices, are also listed in this chapter.

Chapter 2. Notation and Definitions—This chapter lists all the notations that were used in the Code and Commentary. The various terminologies used in the Code are also defined in this chapter.

Chapter 3. Quality and Construction—This chapter provides the details of a quality assurance program by which the quality of masonry construction is monitored. This chapter is very important because masonry design provisions in the Code are valid when the quality of masonry construction meets or exceeds that described in the specification.

Chapter 4. General Analysis and Design Considerations— In this chapter requirements of design loads, structural analysis procedures, material properties, and section properties are provided to transfer forces safely from the point of application to the final point of resistance. The chapter also explicitly emphasizes that masonry walls shall not be connected to structural frames unless the connections and walls are designed to resist design interconnecting forces and to accommodate calculated deflections.

Chapter 5. Structural Elements—This chapter provides the requirements for the design of masonry assemblies, beams, columns, pilasters, and corbels.

Chapter 6. Reinforcement, Metal Accessories, and Anchor Bolts—The requirements for steel reinforcement, metal accessories and anchor bolts are provided in this chapter. Requirements of reinforcement and metal accessories include requirements related to embedment, size of reinforcement, placement of reinforcement, protection of reinforcement and metal accessories, standard hooks, and bend diameter for reinforcing bars. The requirements for headed and bent-bar anchor bolts are also provided under the heading of Anchor Bolts.

Chapter 7. Seismic Design Requirements—This chapter provides requirements for the design and construction of masonry assigned to seismic loads. The requirements of this chapter do not apply to the design or detailing of masonry veneers or glass unit masonry systems. Seismic requirements for masonry veneers are provided in Chapter 12. Glass unit masonry systems, by definition and design, are isolated, non-load-bearing elements and therefore cannot be used to resist seismic loads other than those induced by their own mass.

Chapter 8. Allowable Stress Design of Masonry—This chapter provides requirements for allowable stress design of masonry, in which the calculated stresses resulting from nominal loads must not exceed permissible masonry and steel stresses.

Chapter 9. Strength Design of Masonry—This chapter provides minimum requirements for strength design of masonry, in which internal forces resulting from the application of factored loads must not exceed design strength (nominal member strength reduced by a strength-reduction factor ϕ).

Chapter 10. Prestressed Masonry—Prestressed Masonry requirements are omitted from the current version of the code as they may not be of practical use in Saudi Arabia.

Chapter 11. Strength Design of Autoclaved Aerated Concrete (AAC) Masonry— This chapter provides minimum requirements for the design of AAC masonry. According to this chapter, AAC masonry members shall be proportioned so that the design strength equals or exceeds the required strength. Design strength is the nominal strength multiplied by the strength-reduction factor, ϕ and required strength shall be determined in accordance with the strength design load combinations of SBC 301.

Chapter 12. Veneer—A masonry wythe that provides the exterior finish of a wall system and transfers out-of-plane load directly to a backing, but is not considered to add strength or stiffness to the wall system, is called Veneer. There are two common types of masonry veneer: (1) anchored masonry veneer, and (2) adhered masonry veneer. This chapter provides requirements for design and detailing of both types of masonry veneer.

Chapter 13. Glass Unit Masonry—Glass unit masonry is used as a non-load-bearing element in interior and exterior walls, partitions, window openings, and as an architectural feature. This chapter provides requirements for the empirical design of glass unit masonry as non-load-bearing elements in exterior or interior walls.

Chapter 14. Masonry Partition Walls—This chapter provides requirements for the design of masonry partition walls. These design requirements are prescriptive in nature and based on the condition that vertical loads are reasonably centered on the walls and lateral loads are limited. Members not participating in the lateral-force-resisting system of a building may be designed by the prescriptive provisions of this chapter even though the lateral-force-resisting system is designed under another chapter.

Appendix A. Empirical Design of Masonry—This appendix provides requirements for the empirical design of masonry.

Appendix B. Design of Masonry Infill—Appendix B provides minimum requirements for the structural design of concrete masonry, clay masonry, and AAC masonry infills, either non-participating or participating.

Appendix C. Limit Design Method—This appendix provides alternative design provisions, called limit state design provisions, for special reinforced masonry shear walls subjected to in-plane seismic loading. The limit design is considered to be particularly useful for perforated wall configurations for which a representative yield mechanism can be determined.

Appendix D. Masonry Fireplaces—This appendix provides the provisions for the design and construction of masonry fireplaces, consisting of concrete or masonry (referred to as "masonry fireplaces").

Appendix E. Masonry Heaters— Masonry heaters are appliances designed to absorb and store heat from a relatively small fire and to radiate that heat into the building interior. They are thermally more efficient than traditional fireplaces because of their design. Interior passageways through the heater allow hot exhaust gases from the fire to transfer heat into the masonry, which then radiates into the building. Masonry heaters shall be designed and installed in accordance with this appendix.

Appendix F. Masonry Chimneys—The construction of masonry chimneys consisting of solid masonry units, hollow masonry units grouted solid, stone or concrete shall be in

TABLE OF CONTENTS

PART 1—GENERAL

CHAPTER 1—GENERAL REQUIREMENTS

1.1—Scope

1.1.1 Minimum requirements

The Saudi Building Code for Masonry Structures referred to as SBC 305 provides minimum requirements for the structural design and construction of masonry elements consisting of masonry units bedded in mortar.

1.1.2 Governing building code

SBC 305 shall govern in matters pertaining to structural design and construction of masonry elements. In areas without a legally adopted building code, this Code defines the minimum acceptable standards of design and construction practice.

1.1.3 Unit information

The equations in this document are for use with the specified mm-newton units

1.2—Contract documents and calculations

1.2.1 Show all Code-required drawing items on the project drawings, including:

(a) Name and date of issue of Code and supplement to which the design conforms.
(b) Loads used for the design of masonry structures.
(c) Specified compressive strength of masonry at stated ages or stages of construction for which masonry is designed, for each part of the structure, except for masonry designed in accordance with Part 4 or Appendix A.
(d) Size and location of structural elements.
(e) Details of anchorage of masonry to structural members, frames, and other construction, including the type, size, and location of connectors.
(f) Details of reinforcement, including the size, grade, type, lap splice length, and location of reinforcement.
(g) Reinforcing bars to be welded and welding requirements.
(h) Provision for dimensional changes resulting from elastic deformation, creep, shrinkage, temperature, and moisture.
(i) Size and permitted location of conduits, pipes, and sleeves.

1.2.2 Each portion of the structure shall be designed based on the specified compressive strength of masonry for that part of the structure, except for portions designed in accordance with Part 4 or Appendix A.

1.2.3 The contract documents shall be consistent with design assumptions.

1.2.4 Contract documents shall specify the minimum level of quality assurance as defined in Section 3.1, or shall include an itemized quality assurance program that equals or exceeds the requirements of Section 3.1.

1.3—Approval of special systems of design or construction

Sponsors of any system of design or construction within the scope of this Code, the adequacy of which has been shown by successful use or by analysis or test, but that does not conform to or is not addressed by this Code, shall have the right to present the data on which their design is based to a board of examiners appointed by the building official. The board shall be composed of licensed design professionals and shall have authority to investigate the submitted data, require tests, and formulate rules governing design and construction of such systems to meet the intent of this Code. The rules, when approved and promulgated by the building official, shall be of the same force and effect as the provisions of this Code.

1.4—Standards cited in this Code

Standards of the American Concrete Institute, the American Society of Civil Engineers, ASTM International, the American Welding Society, and The Masonry Society cited in this Code are listed below with their serial designations, including year of adoption or revision, and are declared to be part of this Code as if fully set forth in this document.

TMS 602-13/ACI 530.1-13/ASCE 6-13 — Specification for Masonry Structures

SBC-301 — Minimum Design Loads for Buildings and Other Structures

ASTM A416/A416M-12 — Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete

ASTM A421/A421M-10 — Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete

ASTM A706/A706M-09b — Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement

ASTM C34-12 — Standard Specification for Structural Clay Load-Bearing Wall Tile

ASTM C140-12a — Standard Test Methods for Sampling and Testing Concrete Masonry Units and Related Units

ASTM C426-10 — Standard Test Method for Linear Drying Shrinkage of Concrete Masonry Units

ASTM C476-10 — Standard Specification for Grout for Masonry

ASTM C482-02 (2009) — Standard Test Method for Bond Strength of Ceramic Tile to Portland Cement Paste

ASTM C1006-07 — Standard Test Method for Splitting Tensile Strength of Masonry Units

ASTM C1611/C1611M-09bel — Standard Test Method for Slump Flow of Self-Consolidating Concrete

ASTM C1693-11 — Standard Specification for Autoclaved Aerated Concrete (AAC)

ASTM E111-04 (2010) — Standard Test Method for Young's Modulus, Tangent Modulus, and Chord Modulus

ASTM E488-96 (2003) Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements

AWS D 1.4/D1.4M: 2011 — Structural Welding Code — Reinforcing Steel

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A_b cross-sectional area of an anchor bolt, mm^2
A_{br} bearing area, mm^2
A_g gross cross-sectional area of a member, mm^2
A_n net cross-sectional area of a member, mm^2
A_{nv} net shear area, mm^2
A_{pt} projected tension area on masonry surface of a right circular cone, mm^2
A_{pv} projected shear area on masonry surface of one-half of a right circular cone, mm^2
A_s area of nonprestressed longitudinal tension reinforcement, mm^2
A_{sc} area of reinforcement placed within the lap, near each end of the lapped reinforcing bars and transverse to them, mm^2
A_{st} total area of laterally tied longitudinal reinforcing steel, mm^2
A_v cross-sectional area of shear reinforcement, mm^2
A_1 loaded area, mm^2
A_2 supporting bearing area, mm^2
a depth of an equivalent compression stress block at nominal strength, mm
B_a allowable axial load on an anchor bolt, N
B_{ab} allowable axial tensile load on an anchor bolt when governed by masonry breakout, N
B_{an} nominal axial strength of an anchor bolt, N
B_{anb} nominal axial tensile strength of an anchor bolt when governed by masonry breakout, N
B_{anp} nominal axial tensile strength of an anchor bolt when governed by anchor pullout, N
B_{ans} nominal axial tensile strength of an anchor bolt when governed by steel yielding, N
B_{ap} allowable axial tensile load on an anchor bolt when governed by anchor pullout, N
B_{as} allowable axial tensile load on an anchor bolt when governed by steel yielding, N
B_v allowable shear load on an anchor bolt, N
B_{vb} allowable shear load on an anchor bolt when governed by masonry breakout, N
B_{vc} allowable shear load on an anchor bolt when governed by masonry crushing, N
B_{vn} nominal shear strength of an anchor bolt, N
B_{vnb} nominal shear strength of an anchor bolt when governed by masonry breakout, N
B_{vnc} nominal shear strength of an anchor bolt when governed by masonry crushing, N
B_{vnpry} nominal shear strength of an anchor bolt when governed by anchor pryout, N
B_{vns} nominal shear strength of an anchor bolt when governed by steel yielding, N
B_{vpry} allowable shear load on an anchor bolt when governed by anchor pryout, N
B_{vs} allowable shear load on an anchor bolt when governed by steel yielding, N
b width of section, mm
b_a total applied design axial force on an anchor bolt, N
b_{af} factored axial force in an anchor bolt, N
b_v total applied design shear force on an anchor bolt, N
b_{vf} factored shear force in an anchor bolt, N
b_w width of wall beam, mm
C_d deflection amplification factor
c distance from the fiber of maximum compressive strain to the neutral axis, mm
D dead load or related internal moments and forces
d distance from extreme compression fiber to centroid of tension reinforcement, mm
d_b nominal diameter of reinforcement or anchor bolt, mm
d_v actual depth of a member in direction of shear considered, mm
E load effects of earthquake or related internal moments and forces
E_{AAC} modulus of elasticity of AAC masonry in compression, MPa
E_{bb} modulus of elasticity of bounding beams, MPa
E_{bc} modulus of elasticity of bounding columns, MPa
E_m modulus of elasticity of masonry in compression, MPa
E_s modulus of elasticity of steel, MPa
E_v modulus of rigidity (shear modulus) of masonry, MPa
e eccentricity of axial load, mm
e_b projected leg extension of bent-bar anchor, measured from inside edge of anchor at

bend to farthest point of anchor in the plane of the hook, mm

e_u eccentricity of P_{uf}, mm

F_a allowable compressive stress available to resist axial load only, MPa

F_b allowable compressive stress available to resist flexure only, MPa

F_s allowable tensile or compressive stress in reinforcement, MPa

F_v allowable shear stress, MPa

F_{vm} allowable shear stress resisted by the masonry, MPa

F_{vs} allowable shear stress resisted by the shear reinforcement, MPa

f_a calculated compressive stress in masonry due to axial load only, MPa

f_b calculated compressive stress in masonry due to flexure only, MPa

f'_{AAC} specified compressive strength of AAC masonry, MPa

f'_g specified compressive strength of grout, MPa

f'_m specified compressive strength of clay masonry or concrete masonry, MPa

f_r modulus of rupture, MPa

f_{rAAC} modulus of rupture of AAC, MPa

f_s calculated tensile or compressive stress in reinforcement, MPa

f_{se} effective stress in prestressing tendon after all prestress losses have occurred, MPa

f_{tAAC} splitting tensile strength of AAC as determined in accordance with ASTM C1006, MPa

f_v calculated shear stress in masonry, MPa

f_y specified yield strength of steel for reinforcement and anchors, MPa

h effective height of column, wall, or pilaster, mm

h_{inf} vertical dimension of infill, mm

h_w height of entire wall or of the segment of wall considered, mm

I_{bb} moment of inertia of bounding beam for bending in the plane of the infill, mm^4

I_{bc} moment of inertia of bounding column for bending in the plane of the infill, mm^4

I_{cr} moment of inertia of cracked cross-sectional area of a member, mm^4

I_{eff} effective moment of inertia, mm^4

I_g moment of inertia of gross cross-sectional area of a member, mm^4

I_n moment of inertia of net cross-sectional area of a member, mm^4

j ratio of distance between centroid of flexural compressive forces and centroid of tensile forces to depth, d

K dimension used to calculate reinforcement development, mm

K_{AAC} dimension used to calculate reinforcement development for AAC masonry, mm

k_c coefficient of creep of masonry, per MPa

k_e coefficient of irreversible moisture expansion of clay masonry

k_m coefficient of shrinkage of concrete masonry

k_t coefficient of thermal expansion of masonry per degree Celsius

L live load or related internal moments and forces

l clear span between supports, mm

l_b effective embedment length of headed or bent anchor bolts, mm

l_{be} anchor bolt edge distance, mm

l_d development length or lap length of straight reinforcement, mm

l_e equivalent embedment length provided by standard hooks measured from the start of the hook (point of tangency), mm

l_{eff} effective span length for a deep beam, mm

l_{inf} plan length of infill, mm

l_w length of entire wall or of the segment of wall considered in direction of shear force, mm

M maximum moment at the section under consideration, N-mm

M_a maximum moment in member due to the applied unfactored loading for which deflection is calculated, N-mm

M_{cr} nominal cracking moment strength, N-mm

M_n nominal moment strength, N-mm

M_{ser} service moment at midheight of a member, including P-delta effects, N-mm

M_u factored moment, magnified by second-order effects where required by the code, N-mm

$M_{u,0}$ factored moment from first-order analysis, N-mm

n modular ratio, E_s/E_m

N_u factored compressive force acting normal to shear surface that is associated with the V_u loading combination case under consideration, N

N_v compressive force acting normal to shear surface, N

P Axial load, N

P_a allowable axial compressive force in a reinforced member, N

P_e Euler buckling load, N
P_n nominal axial strength, N
P_u factored axial load, N
P_{uf} factored load from tributary floor or roof areas, N
P_{uw} factored weight of wall area tributary to wall section under consideration, N
Q first moment about the neutral axis of an area between the extreme fiber and the plane at which the shear stress is being calculated, mm^3
Q_E the effect of horizontal seismic (earthquake induced) forces
$q_{n\,inf}$ nominal out-of-plane flexural capacity of infill per unit area, Pa
q_z velocity pressure determined in accordance with SBC 301, kPa
R response modification coefficient
r radius of gyration, mm
S snow load or related internal moments and forces
S_n section modulus of the net cross-sectional area of a member, mm^3
s spacing of reinforcement, mm
s_l total linear drying shrinkage of concrete masonry units determined in accordance with ASTM C426
t nominal thickness of member, mm
t_{inf} specified thickness of infill, mm
$t_{net\,inf}$ net thickness of infill, mm
t_{sp} specified thickness of member, mm
v shear stress, MPa
V shear force, N
V_{lim} limiting base-shear strength, N
V_{nAAC} nominal shear strength provided by AAC masonry, N
V_n nominal shear strength, N
$V_{n\,inf}$ nominal horizontal in-plane shear strength of infill, N
V_{nm} nominal shear strength provided by masonry, N
V_{ns} nominal shear strength provided by shear reinforcement, N
V_u factored shear force, N
V_{ub} base-shear demand, N
W wind load or related internal moments and forces
W_s dimension of the structural wall strip defined in Sections 14.3.2 and A.5.1 and shown in
Figure 14.1 and Figure A.2.
W_T dimension of the tributary length of wall, defined in Sections 14.3.2 and A.5.1 and shown in
Figure 14.1 and Figure A.2.
w_{inf} width of equivalent strut, mm
w_{strut} horizontal projection of the width of the diagonal strut, mm
w_u out-of-plane factored uniformly distributed load, N/mm
z internal lever arm between compressive and tensile forces in a deep beam, mm
α_{arch} horizontal arching parameter for infill, N$^{0.25}$
β_{arch} vertical arching parameter for infill, N$^{0.25}$
β_b ratio of area of reinforcement cut off to total area of tension reinforcement at a section
γ reinforcement size factor
γ_g grouted shear wall factor
Δ calculated story drift, mm
Δ_a allowable story drift, mm
δ moment magnification factor
δ_{ne} displacements calculated using code-prescribed seismic forces and assuming elastic behavior, mm
δ_s horizontal deflection at midheight under allowable stress design load combinations, mm
δ_u Deflection due to factored loads, mm
ε_{cs} drying shrinkage of AAC
ε_{mu} maximum usable compressive strain of masonry
ξ lap splice confinement reinforcement factor
θ_{strut} angle of infill diagonal with respect to the horizontal, degrees
λ_{strut} characteristic stiffness parameter for infill, mm^{-1}
μ_{AAC} coefficient of friction of AAC
ρ reinforcement ratio
ρ_{max} maximum flexural tension reinforcement ratio
ϕ strength-reduction factor
ψ magnification factor for second-order effects

2.2—Definitions

Anchor — Metal rod, wire, or strap that secures masonry to its structural support.

Anchor pullout — Anchor failure defined by the anchor sliding out of the material in which it is embedded without breaking out a substantial portion of the surrounding material.

Area, gross cross-sectional — The area delineated by the out-to-out dimensions of masonry in the plane under consideration.

Area, net cross-sectional — The area of masonry units, grout, and mortar crossed by the plane under consideration based on out-to-out dimensions.

Area, net shear — The net area of the web of a shear element.

Autoclaved aerated concrete — Low-density cementitious product of calcium silicate hydrates, whose material specifications are defined in ASTM C1693.

Autoclaved aerated concrete (AAC) — Masonry - Autoclaved Aerated concrete units manufactured without reinforcement, set on a mortar leveling bed, bonded with thin-bed mortar, placed with or without grout, and placed with or without reinforcement.

Backing — Wall or surface to which veneer is attached.

Bed joint — The horizontal layer of mortar on which a masonry unit is laid.

Bond beam — a horizontal, sloped, or stepped element that is fully grouted, has longitudinal bar reinforcement, and is constructed within a masonry wall.

Bounding frame — The columns and upper and lower beams or slabs that surround masonry infill and provide structural support.

Building official — The officer or other designated authority charged with the administration and enforcement of this Code, or the building official's duly authorized representative.

Cavity wall — A masonry wall consisting of two or more wythes, at least two of which are separated by a continuous air space; air space(s) between wythes may contain insulation; and separated wythes must be connected by wall ties.

Collar joint — Vertical longitudinal space between wythes of masonry or between masonry wythe and backup construction, which is permitted to be filled with mortar or grout.

Column — A structural member, not built integrally into a wall, designed primarily to resist compressive loads parallel to its longitudinal axis and subject to dimensional limitations.

Composite action — Transfer of stress between components of a member designed so that in resisting loads, the combined components act together as a single member.

Composite masonry — Multiwythe masonry members with wythes bonded to produce composite action.

Compressive strength of masonry — Maximum compressive force resisted per unit of net cross-sectional area of masonry, determined by testing masonry prisms or a function of individual masonry units, mortar, and grout, in accordance with the provisions of TMS 602/ACI 530.1/ ASCE 6.

Connector — A mechanical device for securing two or more pieces, parts, or members together, including anchors, wall ties, and fasteners.

Contract documents — Documents establishing the required work, and including in particular, the project drawings and project specifications.

Corbel — A projection of successive courses from the face of masonry.

Cover, grout — thickness of grout surrounding the outer surface of embedded reinforcement, anchor, or tie.

Cover, masonry — thickness of masonry units, mortar, and grout surrounding the outer surface of embedded reinforcement, anchor, or tie.

Cover, mortar — thickness of mortar surrounding the outer surface of embedded reinforcement, anchor, or tie.

Deep beam — A beam that has an effective span-to depth ratio, l_{eff}/d_v, less than 3 for a continuous span and less than 2 for a simple span.

Depth — The dimension of a member measured in the plane of a cross section perpendicular to the neutral axis.

Design story drift — The difference of deflections at the top and bottom of the story under consideration, taking into account the possibility of inelastic deformations as defined in SBC 301. In the equivalent lateral force method, the story drift is calculated by multiplying the deflections determined from an elastic analysis by the appropriate deflection amplification factor, C_d from SBC 301.

Design strength — The nominal strength of an element multiplied by the appropriate strength-reduction factor.

Diaphragm — A roof or floor system designed to transmit lateral forces to shear walls or other lateral-force resisting elements.

Dimension, nominal — The specified dimension plus an allowance for the joints with which the units are to be laid. Nominal dimensions are usually stated in whole numbers nearest to the specified dimensions.

Dimensions, specified — Dimensions specified for the manufacture or construction of a unit, joint, or element.

Effective height — Clear height of a member between lines of support or points of support and used for calculating the slenderness ratio of a member. Effective height for unbraced members shall be calculated.

Foundation pier — A vertical foundation member, not built integrally into a foundation wall, empirically designed to support gravity loads and subject to dimensional limitations.

Glass unit masonry — Masonry composed of glass units bonded by mortar.

Grout — (1) A plastic mixture of cementitious materials, aggregates, and water, with or without admixtures, initially produced to pouring consistency without segregation of the constituents during placement.
(2) The hardened equivalent of such mixtures.

Grout, self-consolidating — A highly fluid and stable grout typically with admixtures, that remains homogeneous when placed and does not require puddling or vibration for consolidation.

Head joint — Vertical mortar joint placed between masonry units within the wythe at the time the masonry units are laid.

Header (bonder) — A masonry unit that connects two or more adjacent wythes of masonry.

Infill — Masonry constructed within the plane of, and bounded by, a structural frame.

Infill, net thickness — Minimum total thickness of the net cross-sectional area of an infill.

Infill, non-participating — Infill designed so that in-plane loads are not imparted to it from the bounding frame.

Infill, participating — Infill designed to resist in-plane loads imparted to it by the bounding frame.

Inspection, continuous — The Inspection Agency's full-time observation of work by being present in the area where the work is being performed.

Inspection, periodic — The Inspection Agency's part-time or intermittent observation of work during construction by being present in the area where the work has been or is being performed, and observation upon completion of the work.

Licensed design professional — An individual who is licensed to practice design as defined by the statutory requirements of the professional licensing laws of the state or jurisdiction in which the project is to be constructed and who is in responsible charge of the design; in other documents, also referred to as registered design professional.

Load, dead — Dead weight supported by a member, as defined by SBC 301.

Load, live — Live load specified by SBC 301.

Load, service — Load specified by SBC 301.

Longitudinal reinforcement — Reinforcement placed parallel to the longitudinal axis of the member.

Masonry breakout — Anchor failure defined by the separation of a volume of masonry, approximately conical in shape, from the member.

Masonry, partially grouted — Construction in which designated cells or spaces are filled with grout, while other cells or spaces are ungrouted.

Masonry unit, hollow — A masonry unit with net cross-sectional area of less than 75 percent of its gross cross-sectional area when measured in any plane parallel to the surface containing voids.

Masonry unit, solid — A masonry unit with net cross-sectional area of 75 percent or more of its gross cross-sectional area when measured in every plane parallel to the surface containing voids.

Modulus of elasticity — Ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material.

Modulus of rigidity — Ratio of unit shear stress to unit shear strain for unit shear stress below the proportional limit of the material.

Nominal strength — The strength of an element or cross section calculated in accordance with the requirements and assumptions of the strength design methods of these provisions before application of strength-reduction factors.

Partition wall — An interior wall without structural function.

Pier —A reinforced, vertically spanning portion of a wall next to an opening, designed using strength design, and subject to dimensional limitations.

Prism — An assemblage of masonry units and mortar, with or without grout, used as a test specimen for determining properties of the masonry.

Project drawings — The drawings that, along with the project specifications, complete the descriptive information for constructing the work required by the contract documents.

Project specifications — The written documents that specify requirements for a project in accordance with the service parameters and other specific criteria established by the owner or the owner's agent.

Quality assurance — The administrative and procedural requirements established by the contract documents to assure that constructed masonry is in compliance with the contract documents.

Reinforcement — Nonprestressed steel reinforcement.

Required strength — The strength needed to resist factored loads.

Running bond — The placement of masonry units so that head joints in successive courses are horizontally offset at least one-quarter the unit length.

Shear wall — A wall, load-bearing or non-load-bearing, designed to resist lateral forces acting in the plane of the wall (sometimes referred to as a vertical diaphragm).

Shear wall, detailed plain (unreinforced) AAC masonry — An AAC masonry shear wall designed to resist lateral forces while neglecting stresses in reinforcement, although provided with minimum reinforcement and connections.

Shear wall, detailed plain (unreinforced) masonry — A masonry shear wall designed to resist lateral forces while neglecting stresses in reinforcement, although provided with minimum reinforcement and connections.

Shear wall, intermediate reinforced masonry — A masonry shear wall designed to resist lateral forces while considering stresses in reinforcement and to satisfy specific minimum reinforcement and connection requirements.

Shear wall, ordinary plain (unreinforced) AAC masonry — An AAC masonry shear wall designed to resist lateral forces while neglecting stresses in reinforcement, if present.

Shear wall, ordinary plain (unreinforced) masonry — A masonry shear wall designed to resist lateral forces while neglecting stresses in reinforcement, if present.

Shear wall, ordinary reinforced AAC masonry — An AAC masonry shear wall designed to resist lateral forces while considering stresses in reinforcement and satisfying prescriptive reinforcement and connection requirements.

Shear wall, ordinary reinforced masonry — A masonry shear wall designed to resist lateral forces while considering stresses in reinforcement and satisfying prescriptive reinforcement and connection requirements.

Shear wall, special reinforced masonry — A masonry shear wall designed to resist lateral forces while considering stresses in reinforcement and to satisfy special reinforcement and connection requirements.

Slump flow — The circular spread of plastic self-consolidating grout, which is evaluated in accordance with ASTM C1611/C1611M.

Special boundary elements — in walls that are designed to resist in-plane load, end regions that are strengthened by reinforcement and are detailed to meet specific requirements, and may or may not be thicker than the wall.

Specified compressive strength of AAC masonry, f'_{AAC} — Minimum compressive strength, expressed as force per unit of net cross-sectional area, required of the AAC masonry used in construction by the contract documents, and upon which the project design is based. Whenever the quantity f'_{AAC} is under the radical sign, the square root of numerical value only is intended and the result has units of MPa.

Specified compressive strength of masonry, f'_m — Minimum compressive strength, expressed as force per unit of net cross-sectional area, required of the masonry used in construction by the contract documents, and upon which the project design is based. Whenever the quantity f'_m is under the radical sign, the square root of numerical value only is intended and the result has units of MPa.

Stirrup — Reinforcement used to resist shear in a flexural member.

Stone masonry — Masonry composed of field, quarried, or cast stone units bonded by mortar.

Stone masonry, ashlar — Stone masonry composed of rectangular units having sawed, dressed, or squared bed surfaces and bonded by mortar.

Stone masonry, rubble — Stone masonry composed of irregular-shaped units bonded by mortar.

Strength-reduction factor, ϕ — the factor by which the nominal strength is multiplied to obtain the design strength.

Thin-bed mortar — Mortar for use in construction of AAC unit masonry whose joints shall not be less than 1.5 mm.

Tie, lateral — Loop of reinforcing bar or wire enclosing longitudinal reinforcement.

Tie, wall — Metal connector that connects wythes of masonry walls together.

Transverse reinforcement — Reinforcement placed perpendicular to the longitudinal axis of the member.

Unreinforced (plain) masonry — Masonry in which the tensile resistance of masonry is taken into consideration and the resistance of reinforcing steel, if present, is neglected.

Veneer, adhered — Masonry veneer secured to and supported by the backing through adhesion.

Veneer, anchored — Masonry veneer secured to and supported laterally by the backing through anchors and supported vertically by the foundation or other structural elements.

Veneer, masonry — A masonry wythe that provides the exterior finish of a wall system and transfers out-of-plane load directly to a backing, but is not considered to add strength or stiffness to the wall system.

Visual stability index (VSI) — an index, defined in ASTM C1611/C1611M, that qualitatively indicates the stability of self-consolidating grout

Wall — A vertical element with a horizontal length to thickness ratio greater than 3, used to enclose space.

Wall, load-bearing — Wall supporting vertical loads greater than 3000 N/m in addition to its own weight.

Wall, masonry bonded hollow — a multiwythe wall built with masonry units arranged to provide an air space between the wythes and with the wythes bonded together with masonry units.

Width — the dimension of a member measured in the plane of a cross section parallel to the neutral axis.

Wythe — each continuous vertical section of a wall, one masonry unit in thickness.

CHAPTER 3—QUALITY AND CONSTRUCTION

3.1—Quality assurance program

The quality assurance program shall comply with the requirements of this section, depending on the Risk Category, as defined in SBC 301. The quality assurance program shall itemize the requirements for verifying conformance of material composition, quality, storage, handling, preparation, and placement with the requirements of TMS 602/ACI 530.1/ASCE 6.

3.1.1 Level A Quality Assurance

The minimum quality assurance program for masonry in Risk Category I, II, or III structures and designed in accordance with Part 4 or Appendix A shall comply with Table 3.1.

3.1.2 Level B Quality Assurance

3.1.2.1 The minimum quality assurance program for masonry in Risk Category IV structures and designed in accordance with Chapter 12 or Chapter 13 shall comply with Table 3.2.

3.1.2.2 The minimum quality assurance program for masonry in Risk Category I, II, or III structures and designed in accordance with chapters other than those in Part 4 or Appendix A shall comply with Table 3.2.

3.1.3 Level C Quality Assurance

The minimum quality assurance program for masonry in Risk Category IV structures and designed in accordance with chapters other than those in Part 4 or Appendix A shall comply with Table 3.3.

3.1.4 Procedures

The quality assurance program shall set forth the procedures for reporting and review. The quality assurance program shall also include procedures for resolution of noncompliances.

3.1.5 Qualifications

The quality assurance program shall define the qualifications for testing laboratories and for inspection agencies.

3.1.6 Acceptance relative to strength requirements

3.1.6.1 Compliance with f'_m — Compressive strength of masonry shall be considered satisfactory if the compressive strength of each masonry wythe and grouted collar joint equals or exceeds the value of f'_m.

3.1.6.2 Determination of compressive strength — Compressive strength of masonry shall be determined in accordance with the provisions of TMS 602/ACI 530.1/ASCE 6.

3.2—Construction considerations

3.2.1 Grouting, minimum spaces

The minimum dimensions of spaces provided for the placement of grout shall be in accordance with Table 3.4. Grout pours with heights exceeding those shown in Table 3.4, cavity widths, or cell sizes smaller than those permitted in Table 3.4 or grout lift heights exceeding those permitted by Article 3.5 D of TMS 602/ACI 530.1/ ASCE 6 are permitted if the results of a grout demonstration panel show that the grout spaces are filled and adequately consolidated. In that case, the procedures used in constructing the grout demonstration panel shall be the minimum acceptable standard for grouting, and the quality assurance program shall include inspection during construction to verify grout placement.

3.2.2 Embedded conduits, pipes, and sleeves

Conduits, pipes, and sleeves of any material to be embedded in masonry shall be compatible with masonry and shall comply with the following requirements.

3.2.2.1 Conduits, pipes, and sleeves shall not be considered to be structural replacements for the displaced masonry. The masonry design shall consider the structural effects of this displaced masonry.

3.2.2.2 Conduits, pipes, and sleeves in masonry shall be no closer than 3 diameters on center. Minimum spacing of conduits, pipes or sleeves of different diameters shall be determined using the larger diameter.

3.2.2.3 Vertical conduits, pipes, or sleeves placed in masonry columns or pilasters shall not displace more than 2 percent of the net cross section.

3.2.2.4 Pipes shall not be embedded in masonry, unless properly isolated from the masonry, when:

(a) Containing liquid, gas, or vapors at temperature higher than 66°C.
(b) Under pressure in excess of 380 kPa.
(c) Containing water or other liquids subject to freezing.

3.2.3 Separation Joints.

Where concrete abuts structural masonry and the joint between the materials is not designed as a separation joint, the concrete shall be roughened so that the average height of aggregate exposure is 3 mm and shall be bonded to the masonry in accordance with these requirements as if it were masonry. Vertical joints not intended to act as separation joints shall be crossed by horizontal reinforcement as required by Section 5.1.1.2.

TABLES OF CHAPTER 3

Table 3.1: Level A Quality Assurance

MINIMUM VERIFICATION
Prior to construction, verify certificates of compliance used in masonry construction

Table 3.2: Level B Quality Assurance

MINIMUM TESTS				
Verification of Slump flow and Visual Stability Index (VSI) as delivered to the project site in accordance with Specification Article 1.5 B.1.b.3 for self-consolidating grout				
Verification off f'_m and f'_{AAC} in accordance with Specification Article 1.4 B prior to construction, except where specifically exempted by this Code				
MINIMUM SPECIAL INSPECTION				
Inspection Task	Frequency [(a)]		Reference for Criteria	
	Continuous	Periodic	SBC 305	TMS 602/ ACI 530.1/ ASCE 6
1. Verify compliance with the approved submittals		X		Art. 1.5
2. As masonry construction begins, verify that the following are in compliance:				
a. Proportions of site-prepared mortar		X		Art. 2.1, 2.6 A
b. Construction of mortar joints		X		Art. 3.3 B
c. Location of reinforcement, connectors, and anchorages		X		Art. 3.4, 3.6 A
d. Properties of thin-bed mortar for AAC masonry	X[(b)]	X[(c)]		Art. 2.1 C
3. Prior to grouting, verify that the following are in compliance:				
a. Grout space		X		Art. 3.2 D, 3.2 F
b. Grade, type, and size of reinforcement , anchor bolts, and anchorages		X	Sec. 6.1	Art. 2.4, 3.4
c. Placement of reinforcement, connectors, and anchorages		X	Sec. 6.1, 6.2.1, 6.2.6, 6.2.7	Art. 3.2 E, 3.4, 3.6 A
d. Proportions of site-prepared grout.		X		Art. 2.6 B, 2.4 G.1.b
e. Construction of mortar joints		X		Art. 3.3 B

Continued on next page

Table 3.2: Level B Quality Assurance(Continued)

MINIMUM SPECIAL INSPECTION				
Inspection Task	Frequency (a)		Reference for Criteria	
	Continuous	Periodic	SBC 305	TMS 602/ ACI 530.1/ ASCE 6
4. Verify during construction:				
a. Size and location of structural elements		X		Art. 3.3 F
b. Type, size, and location of anchors, including other details of anchorage of masonry to structural members, frames, or other construction		X	Sec. 1.2.1(e), 6.1.4.3, 6.2.1	
c. Welding of reinforcement	X		Sec. 8.1.6.7.2, 9.3.3.4 (c), 11.3.3.4(b)	
d. Preparation, construction, and protection of masonry during cold weather (temperature below 4.4°C or hot weather (temperature above 32.2°C)		X		Art. 1.8 C, 1.8 D
e. Placement of grout	X			Art. 3.5, 3.6 C
f. Placement of AAC masonry units and construction of thin-bed mortar joints	X(b)	X(c)		Art. 3.3 B.9, 3.3 F.1.b
5. Observe preparation of grout specimens, mortar specimens, and/or prisms		X		Art. 1.4 B.2.a.3, 1.4 B.2.b.3, 1.4 B.2.c.3, 1.4 B.3, 1.4 B.4

(a) Frequency refers to the frequency of Special Inspection, which may be continuous during the task listed or periodic during the listed task, as defined in the table.
(b) Required for the first 500 square meters of AAC masonry.
(c) Required after the first 500 square meters of AAC masonry.

Table 3.3: Level C Quality Assurance

MINIMUM TESTS				
Verification of f'_m and f'_{ACC} in accordance with Specification Article 1.4 B prior to construction and for every 500 sq. m during construction				
Verification of proportions of materials in premixed or preblended mortar, prestressing grout, and grout other than self-consolidating grout, as delivered to the project site				
Verification of Slump flow and Visual Stability Index (VSI) as delivered to the project site in accordance with Specification Article 1.5 B.1.b.3 for self-consolidating grout				
MINIMUM SPECIAL INSPECTION				
Inspection Task	Frequency [(a)]		Reference for Criteria	
	Continuous	Periodic	SBC 305	TMS 602/ ACI 530.1/ ASCE 6
1. Verify compliance with the approved submittals		X		Art. 1.5
2. Verify that the following are in compliance:				
a. Proportions of site-mixed mortar and grout		X		Art. 2.1, 2.6 A, 2.6 B, 2.6 C, 2.4 G 1.b
b. Grade, type, and size of reinforcement, anchor bolts, and anchorages		X	Sec. 6.1	Art. 2.4, 3.4
c. Placement of masonry units and construction of mortar joints		X		Art. 3.3 B
d. Placement of reinforcement, connectors, and anchorages	X		Sec. 6.1, 6.2.1, 6.2.6, 6.2.7	Art 3.2 E, 3.4, 3.6 A
e. Grout space prior to grouting	X			Art. 3.2 D, 3.2 F
f. Placement of grout	X			Art. 3.5, 3.6 C
g. Size and location of structural elements		X		Art. 3.3 F
h. Type, size, and location of anchors including other details of anchorage of masonry to structural members, frames, or other construction	X		Sec. 1.2.1(e), 6.1.4.3, 6.2.1	
i. Welding of reinforcement	X		Sec. 8.1.6.7.2, 9.3.3.4 (c), 11.3.3.4(b)	
j. Preparation, construction, and protection of masonry during cold weather (temperature below 5°C or hot weather (temperature above 32°C)		X		Art. 1.8 C, 1.8 D
k. Placement of AAC masonry units and construction of thin-bed mortar joints	X			Art. 3.3 B.9, 3.3 F.1.b
l. Properties of thin-bed mortar for AAC masonry	X			Art. 2.1 C.1
3. Observe preparation of grout specimens, mortar specimens, and/or prisms	X			Art. 1.4 B.2.a.3, 1.4 B.2.b.3, 1.4 B.2.c.3, 1.4 B.3, 1.4 B.4

(a) Frequency refers to the frequency of Special Inspection, which may be continuous during the task listed or periodic during the listed task, as defined in the table.

Table 3.4: Grout space requirements

Grout type[1]	Maximum grout pour height, m	Minimum clear width of grout space,[2,3] mm	Minimum clear grout space dimensions for grouting cells of hollow units,[3,4, 5] mm × mm
Fine	0.30	20	38×51
Fine	1.60	50	51 × 76
Fine	3.80	60	63 × 76
Fine	7.30	75	76 × 76
Coarse	0.30	38	38 × 76
Coarse	1.60	50	63 × 76
Coarse	3.80	60	76 × 76
Coarse	7.30	75	76 × 102

[1] Fine and coarse grouts are defined in ASTM C476.

[2] For grouting between masonry wythes.

[3] Minimum clear width of grout space and minimum clear grout space dimension are the net dimension of the space determined by subtracting masonry protrusions and the diameters of horizontal bars from the as-designed cross-section of the grout space. Grout type and maximum grout pour height shall be specified based on the minimum clear space.

[4] Area of vertical reinforcement shall not exceed 6 percent of the area of the grout space.

[5] Minimum grout space dimension for AAC masonry units shall be 76 mm × 76 mm or a 76 mm diameter cell.

PART 2—DESIGN REQUIREMENT

CHAPTER 4—GENERAL ANALYSIS AND DESIGN CONSIDERATIONS

4.1—Loading

4.1.1 General

Masonry shall be designed to resist applicable loads. A continuous load path or paths, with adequate strength and stiffness, shall be provided to transfer forces from the point of application to the final point of resistance.

4.1.2 Load provisions

Design loads shall be in accordance with the SBC 301, with such live load reductions as are permitted in SBC 301, except as noted in this Code.

4.1.3 Lateral load resistance

Buildings shall be provided with a structural system designed to resist wind and earthquake loads and to accommodate the effect of the resulting deformations.

4.1.4 Load transfer at horizontal connections

4.1.4.1 Walls, columns, and pilasters shall be designed to resist loads, moments, and shears applied at intersections with horizontal members.

4.1.4.2 Effect of lateral deflection and translation of members providing lateral support shall be considered.

4.1.4.3 Devices used for transferring lateral support from members that intersect walls, columns, or pilasters shall be designed to resist the forces involved.

4.1.5 Other effects

Consideration shall be given to effects of forces and deformations due to prestressing, vibrations, impact, shrinkage, expansion, temperature changes, creep, unequal settlement of supports, and differential movement.

4.1.6 Lateral load distribution

Lateral loads shall be distributed to the structural system in accordance with member stiffnesses and shall comply with the requirements of this section.

4.1.6.1 Flanges of intersecting walls designed in accordance with Section 5.1.1.2 shall be included in stiffness determination.

4.1.6.2 Distribution of load shall be consistent with the forces resisted by foundations.

4.1.6.3 Distribution of load shall include the effect of horizontal torsion of the structure due to eccentricity of wind or seismic loads resulting from the non-uniform distribution of mass.

4.2—Material properties

4.2.1 General

Unless otherwise determined by test, the following moduli and coefficients shall be used in determining the effects of elasticity, temperature, moisture expansion, shrinkage, and creep.

4.2.2 Elastic moduli

4.2.2.1 Steel reinforcement — Modulus of elasticity of steel reinforcement shall be taken as:

$$E_s = 200{,}000 \text{ MPa} \qquad \text{Equation 4-1}$$

4.2.2.2 Clay and concrete masonry

4.2.2.2.1 The design of clay and concrete masonry shall be based on the following modulus of elasticity values:

$$E_m = 700 f'_m \text{ for clay masonry} \qquad \text{Equation 4-2}$$

$$E_m = 900 f'_m \text{ for concrete masonry} \qquad \text{Equation 4-3}$$

Or the chord modulus of elasticity taken between 0.05 and 0.33 of the maximum compressive strength of each prism determined by test in accordance with the prism test method, Article 1.4 B.3 of TMS 602/ACI 530.1/ASCE 6, and ASTM E111.

4.2.2.2.2 Modulus of rigidity of clay masonry and concrete masonry shall be taken as:

$$E_v = 0.4 E_m \qquad \text{Equation 4-4}$$

4.2.2.3 AAC masonry

4.2.2.3.1 Modulus of elasticity of AAC masonry shall be taken as:

$$E_{AAC} = 888(f'_{AAC})^{0.6} \qquad \text{Equation 4-5}$$

4.2.2.3.2 Modulus of rigidity of AAC masonry shall be taken as:

$$E_v = 0.4E_{AAC} \qquad \text{Equation 4-6}$$

4.2.2.4 Grout — Modulus of elasticity of grout shall be taken as $500f'_g$.

4.2.3 Coefficients of thermal expansion

Material type	k_t (mm/mm/°C)
Clay masonry	7.2×10^{-6}
Concrete masonry	8.1×10^{-6}
AAC masonry	8.1×10^{-6}

4.2.4 Coefficient of moisture expansion

Material type	k_e (mm/mm)
Clay masonry	3×10^{-4}

4.2.5 Coefficients of shrinkage

Material type	k_m
Concrete masonry	$0.5s_l$
Concrete masonry	$0.8\varepsilon_{cs}/100$

where ε_{cs} is determined in accordance with ASTM C1693.

4.2.6 Coefficients of creep

Material type	k_c, per MPa
Clay masonry	0.1×10^{-4}
Concrete masonry	0.36×10^{-4}
AAC masonry	0.72×10^{-4}

4.3—Section properties

4.3.1 Stress calculations

4.3.1.1 Members shall be designed using section properties based on the minimum net cross-sectional area of the member under consideration. Section properties shall be based on specified dimensions.

4.3.1.2 In members designed for composite action, stresses shall be calculated using section properties based on the minimum transformed net cross-sectional area of the composite member. The transformed area concept for elastic analysis, in which areas of dissimilar materials are transformed in accordance with relative elastic moduli ratios, shall apply.

4.3.2 Stiffness

Calculation of stiffness based on uncracked section is permissible. Use of the average net cross-sectional area of the member considered in stiffness calculations is permitted.

4.3.3 Radius of gyration

Radius of gyration shall be calculated using the average net cross-sectional area of the member considered.

4.3.4 Bearing area

The bearing area, A_{br}, for concentrated loads shall not exceed the following:

(a) $A_1\sqrt{A_2/A_1}$
(b) $2A_1$

The area, A_2, is the area of the lower base of the largest frustum of a right pyramid or cone that has the loaded area, A_1, as its upper base, slopes at 45 degrees from the horizontal, and is wholly contained within the support. For walls not laid in running bond, area A_2 shall terminate at head joints.

4.4—Connection to structural frames

Masonry walls shall not be connected to structural frames unless the connections and walls are designed to resist design interconnecting forces and to accommodate calculated deflections.

4.5—Masonry not laid in running bond

For masonry not laid in running bond, the minimum area of horizontal reinforcement shall be 0.00028 multiplied by the gross vertical cross-sectional area of the wall using specified dimensions. Horizontal reinforcement shall be placed at a maximum spacing of 1200 mm on center in horizontal mortar joints or in bond beams.

CHAPTER 5—STRUCTURAL ELEMENTS

5.1—Masonry assemblies

5.1.1 Intersecting walls

5.1.1.1 Wall intersections shall meet one of the following requirements:

(a) Design shall conform to the provisions of Section 5.1.1.2.
(b) Transfer of shear between walls shall be prevented.

5.1.1.2 Design of wall intersection

5.1.1.2.1 Masonry shall be in running bond.

5.1.1.2.2 Flanges shall be considered effective in resisting applied loads.

5.1.1.2.3 The width of flange considered effective on each side of the web shall be the smaller of the actual flange on either side of the web wall or the following:

(a) 6 multiplied by the nominal flange thickness for unreinforced and reinforced masonry, when the flange is in compression
(b) 6 multiplied by the nominal flange thickness for unreinforced masonry, when the flange is in flexural tension
(c) 0.75 multiplied by the floor-to-floor wall height for reinforced masonry, when the flange is in flexural tension.

The effective flange width shall not extend past a movement joint.

5.1.1.2.4 Design for shear, including the transfer of shear at interfaces, shall conform to the requirements of Section 8.2.6; or Section 8.3.5; or Section 9.2.6; or Section 9.3.4.1.2; or Section 11.3.4.1.2.

5.1.1.2.5 The connection of intersecting walls shall conform to one of the following requirements:

(a) At least fifty percent of the masonry units at the interface shall interlock.
(b) Walls shall be anchored by steel connectors grouted into the wall and meeting the following requirements:
    (1) Minimum size: 6.5 mm × 38 mm × 710 mm including 50-mm long, 90-degree bend at each end to form a U or Z shape.
    (2) Maximum spacing: 1200 mm.
(c) Intersecting reinforced bond beams shall be provided at a maximum spacing of 1200 mm on center. The area of reinforcement in each bond beam shall not be less than 210 mm^2/mm multiplied by the vertical spacing of the bond beams in meters. Reinforcement shall be developed on each side of the intersection.

5.1.2 Effective compressive width per bar

5.1.2.1 For masonry not laid in running bond and having bond beams spaced not more than 1200 mm center-to-center, and for masonry laid in running bond, the width of the compression area used to calculate element capacity shall not exceed the least of:

(a) Center-to-center bar spacing.
(b) Six multiplied by the nominal wall thickness.
(c) 1800 mm.

5.1.2.2 For masonry not laid in running bond and having bond beams spaced more than 1200 mm center-to-center, the width of the compression area used to calculate element capacity shall not exceed the length of the masonry unit.

5.1.3 Concentrated loads

5.1.3.1 Concentrated loads shall not be distributed over a length greater than the minimum of the following:

(a) The length of bearing area plus the length determined by considering the concentrated load to be dispersed along a 2 vertical: 1 horizontal line. The dispersion shall terminate at half the wall height, a movement joint, the end of the wall, or an opening, whichever provides the smallest length.
(b) The center-to-center distance between concentrated loads.

5.1.3.2 For walls not laid in running bond, concentrated loads shall not be distributed across head joints. Where concentrated loads acting on such walls are applied to a bond beam, the concentrated load is permitted to be distributed through the bond beam, but shall not be distributed across head joints below the bond beams.

5.1.4 Multiwythe masonry elements

Design of masonry composed of more than one wythe shall comply with the provisions of Section 5.1.4.1, and either 5.1.4.25.1.4.3 or 5.1.4.3.

5.1.4.1 The provisions of Sections 5.1.4.2, and 5.1.4.3 shall not apply to AAC masonry units and glass masonry units.

5.1.4.2 Composite action

5.1.4.2.1 Multiwythe masonry designed for composite action shall have collar joints either:

(a) crossed by connecting headers, or
(b) Filled with mortar or grout and connected by wall ties.

5.1.4.2.2 Headers used to bond adjacent wythes shall meet the requirements of either Section 8.1.4.2 or Section 9.1.7.2 and shall be provided as follows:

(a) Headers shall be uniformly distributed and the sum of their cross-sectional areas shall be at least 4 percent of the wall surface area.
(b) Headers connecting adjacent wythes shall be embedded a minimum of 80 mm in each wythe.

5.1.4.2.3 Wythes not bonded by headers shall meet the requirements of either Section 8.1.4.2 or Section 9.1.7.2 and shall be bonded by non-adjustable ties provided as follows:

Wire size	Minimum number of ties required
WD 4.0	one per 0.25 m^2 of masonry surface area
WD 5.0	one per 0.42 m^2 of masonry surface area

The maximum spacing between ties shall be 900 mm horizontally and 600 mm vertically. The use of rectangular ties to connect masonry wythes of any type of masonry unit shall be permitted. The use of Z ties to connect to a masonry wythe of hollow masonry units shall not be permitted. Cross wires of joint reinforcement shall be permitted to be used instead of ties.

5.1.4.3 Non-composite action — The design of multiwythe masonry for non-composite action shall comply with Sections 5.1.4.3.1 and 5.1.4.3.2:

5.1.4.3.1 Each wythe shall be designed to resist individually the effects of loads imposed on it.

Unless a more detailed analysis is performed, the following requirements shall be satisfied:

(a) Collar joints shall not contain headers, grout, or mortar.
(b) Gravity loads from supported horizontal members shall be resisted by the wythe nearest to the center of span of the supported member. Any resulting bending moment about the weak axis of the masonry element shall be distributed to each wythe in proportion to its relative stiffness.
(c) Lateral loads acting parallel to the plane of the masonry element shall be resisted only by the wythe on which they are applied. Transfer of stresses from such loads between wythes shall be neglected.
(d) Lateral loads acting transverse to the plane of the masonry element shall be resisted by all wythes in proportion to their relative flexural stiffnesses.
(e) Specified distances between wythes shall not exceed 100 mm unless a detailed tie analysis is performed.

5.1.4.3.2 Wythes of masonry designed for non-composite action shall be connected by ties meeting the requirements of Section 5.1.4.2.3 or by adjustable ties. Where the cross wires of joint reinforcement are used as ties, the joint reinforcement shall be ladder-type or tab-type. Ties shall be without cavity drips.

Adjustable ties shall meet the following requirements:

(a) One tie shall be provided for each 0.16 m^2 of masonry surface area.
(b) Horizontal and vertical spacing shall not exceed 400 mm.
(c) Adjustable ties shall not be used when the misalignment of bed joints from one wythe to the other exceeds 30 mm.
(d) Maximum clearance between connecting parts of the tie shall be 1.5 mm.
(e) Pintle ties shall have at least two pintle legs of wire size WD 5.0.

5.2—Beams

Design of beams shall meet the requirements of Section 5.2.1 or Section 5.2.2. Design of beams shall also meet the requirements of Section 8.3, Section 9.3 or Section 11.3. Design requirements for masonry beams shall apply to masonry lintels.

5.2.1 General beam design

5.2.1.1 Span length — Span length shall be in accordance with the following:

5.2.1.1.1 Span length of beams not built integrally with supports shall be taken as the clear span plus depth of beam, but need not exceed the distance between centers of supports.

5.2.1.1.2 For determination of moments in beams that are continuous over supports, span length shall be taken as the distance between centers of supports.

5.2.1.2 Lateral support — The compression face of beams shall be laterally supported at a maximum spacing based on the smaller of:

(a) $32b$
(b) $120b^2/d$

5.2.1.3 Bearing length — Length of bearing of beams on their supports shall be a minimum of 100 mm in the direction of span.

5.2.1.4 Deflections — Masonry beams shall be designed to have adequate stiffness to limit deflections that adversely affect strength or serviceability.

5.2.1.4.1 The calculated deflection of beams providing vertical support to masonry designed in accordance with Section 8.2, Section 9.2, Section 11.2, Chapter 14, or Appendix A shall not exceed $l/600$ under unfactored dead plus live loads.

5.2.1.4.2 Deflection of masonry beams shall be calculated using the appropriate load-deflection relationship considering the actual end conditions. Unless stiffness values are obtained by a more comprehensive analysis, immediate deflections shall be calculated with an effective moment of inertia, I_{eff} as follows:

$$I_{eff} = I_n\left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] \leq I_n \qquad \text{5-1}$$

For continuous beams, I_{eff} shall be permitted to be taken as the average of values obtained from 5-1 for the critical positive and negative moment regions.

For beams of uniform cross-section, I_{eff} shall be permitted to be taken as the value obtained from 5-1 at midspan for simple spans and at the support for cantilevers. For masonry designed in accordance with Chapter 8, the cracking moment, M_{cr}, shall be calculated using the allowable flexural tensile stress taken from Table 8.1 multiplied by a factor of 2.5. For masonry designed in accordance with Chapter 9, the cracking moment, M_{cr}, shall be calculated using the value for the modulus of rupture, f_r, taken from Table 9.1. For masonry designed in accordance with CHAPTER 11, the cracking moment, M_{cr}, shall be calculated using the value for the modulus of rupture, f_{rAAC}, as given by Section 11.1.8.3.

5.2.1.4.3 Deflections of reinforced masonry beams need not be checked when the span length does not exceed 8 multiplied by the effective depth to the reinforcement, d, in the masonry beam.

5.2.2 Deep beams

Design of deep beams shall meet the requirements of Section 5.2.1.2 and 5.2.1.3 in addition to the requirements of 5.2.2.1 through 5.2.2.5.

5.2.2.1 Effective span length — The effective span length l_{eff} shall be taken as the center to center distance between supports or 1.15 multiplied by the clear span. Whichever is smaller.

5.2.2.2 Internal lever arm — Unless determined by a more comprehensive analysis, the internal lever arm, z, shall be taken as:

(a) For simply supported spans.

(1) When $1 \leq \frac{l_{eff}}{d_v} < 2$

$$z = 0.2(l_{eff} + 2d_v) \qquad \text{Equation 5-2a}$$

(2) When $\frac{l_{eff}}{d_v} < 1$

$$z = 0.6l_{eff} \qquad \text{Equation 5-2b}$$

(b) For continuous spans

(1) When $1 \leq \frac{l_{eff}}{d_v} < 3$

$$z = 0.2(l_{eff} + 1.5d_v) \qquad \text{Equation 5-3a}$$

(2) When $\frac{l_{eff}}{d_v} < 1$

$$z = 0.5l_{eff} \qquad \text{Equation 5-3b}$$

5.2.2.3 Flexural reinforcement — Distributed horizontal flexural reinforcement shall be provided in the tension zone of the beam for a depth equal to half of the beam depth, d_v. The maximum spacing of distributed horizontal flexural reinforcement shall not exceed one-fifth of the beam depth, d_v nor 400 mm. Joint reinforcement shall be permitted to be used as distributed horizontal flexural reinforcement in deep beams. Horizontal flexural reinforcement shall be anchored to develop the yield strength of the reinforcement at the face of supports.

5.2.2.4 Minimum shear reinforcement — The following provisions shall apply when shear reinforcement is required in accordance with Section 8.3.5, Section 9.3.4.1.2, or Section 11.3.4.1.2.

(a) The minimum area of vertical shear reinforcement shall be $0.0007bd_v$.
(b) Horizontal shear reinforcement shall have cross-sectional area equal to or greater than one half the area of the vertical shear reinforcement. Such reinforcement shall be equally distributed on both side faces of the beam when the nominal width of the beam is greater than 200 mm.
(c) The maximum spacing of shear reinforcement shall not exceed one-fifth the beam depth, d_v nor 400 mm.

5.2.2.5 Total reinforcement — The sum of the cross-sectional areas of horizontal and vertical reinforcement shall be at least 0.001 multiplied by the gross cross-sectional area, bd_v of the deep beam, using specified dimensions.

5.3—Columns

Design of columns shall meet the requirements of Section 5.3.1 or Section 5.3.2. Design of columns shall also meet the requirements of Section 8.3, or Section 9.3, or Section 11.3.

5.3.1 General column design

5.3.1.1 Dimensional limits — Dimensions shall be in accordance with the following:

(a) The distance between lateral supports of a column shall not exceed 99 multiplied by the least radius of gyration, r.
(b) Minimum side dimension shall be 200 mm nominal.

5.3.1.2 Construction — Columns shall be fully grouted.

5.3.1.3 Vertical reinforcement — Vertical reinforcement in columns shall not be less than $0.0025A_n$ nor exceed $0.04A_n$. The minimum number of bars shall be four.

5.3.1.4 Lateral ties — Lateral ties shall conform to the following:

(a) Vertical reinforcement shall be enclosed by lateral ties at least 6 mm in diameter.
(b) Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 lateral tie bar or wire diameters, or least cross-sectional dimension of the member.
(c) Lateral ties shall be arranged so that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a lateral tie with an included angle of not more than 135 degrees. No bar shall be farther than 150 mm clear on each side along the lateral tie from such a laterally supported bar. Lateral ties shall be placed in either a mortar joint or in grout. Where longitudinal bars are located around the perimeter of a circle, a complete circular lateral tie is permitted. Lap length for circular ties shall be 48 tie diameters.
(d) Lateral ties shall be located vertically not more than one-half lateral tie spacing above the top of footing or slab in any story, and shall be spaced not more than one-half a lateral tie spacing below the lowest horizontal reinforcement in beam, girder, slab, or drop panel above

5.3.2 Lightly loaded columns

Masonry columns used only to support light frame roofs of carports, porches, sheds or similar structures assigned to Seismic Design Category A, B, or C, which are subject to unfactored gravity loads not exceeding 8,900 N acting within the cross-sectional dimensions of the column are permitted to be constructed as follows:

(a) Minimum side dimension shall be 200 mm nominal.
(b) Height shall not exceed 3.50 m.
(c) Cross-sectional area of longitudinal reinforcement shall not be less than 129 mm^2 centered in the column.
(d) Columns shall be fully grouted.

5.4—Pilasters

Walls interfacing with pilasters shall not be considered as flanges, unless the construction requirements of Sections 5.1.1.2.1 and 5.1.1.2.5 are met. When these construction requirements are met, the pilaster's flanges shall be designed in accordance with Sections 5.1.1.2.2 through 5.1.1.2.4.

5.5—Corbels

5.5.1 Load-bearing corbels

Load-bearing corbels shall be designed in accordance with Chapter 8 or Chapter 9.

5.5.2 Non-load-bearing — corbels Non-load-bearing corbels shall be designed in accordance with Chapter 8 or Chapter 9 or or detailed as follows:

(a) Solid masonry units or hollow units filled with mortar or grout shall be used.
(b) The maximum projection beyond the face of the wall shall not exceed:
   (1) One-half the wall thickness for multiwythe walls bonded by mortar or

grout and wall ties or masonry headers, or

(2) One-half the wythe thickness for single wythe walls, masonry bonded hollow walls, multiwythe walls with open collar joints, and veneer walls.

(c) The maximum projection of one unit shall not exceed:

(1) One-half the nominal unit height.

(2) One-third the nominal thickness of the unit or wythe.

(d) The back surface of the corbelled section shall remain within 25 mm of plane.

CHAPTER 6—REINFORCEMENT, METAL ACCESSORIES, AND ANCHOR BOLTS

6.1—Details of reinforcement and metal accessories

6.1.1 Embedment

Reinforcing bars shall be embedded in grout.

6.1.2 Size of reinforcement

6.1.2.1 The maximum size of reinforcement used in masonry shall be Dia 36.

6.1.2.2 The diameter of reinforcement shall not exceed one-half the least clear dimension of the cell, bond beam, or collar joint in which it is placed.

6.1.2.3 Longitudinal and cross wires of joint reinforcement shall have a minimum wire size of WD 4.0 and a maximum wire size of one-half the joint thickness.

6.1.3 Placement of reinforcement

6.1.3.1 The clear distance between parallel bars shall not be less than the nominal diameter of the bars, nor less than 25 mm.

6.1.3.2 In columns and pilasters, the clear distance between vertical bars shall not be less than one and one-half multiplied by the nominal bar diameter, nor less than 38 mm.

6.1.3.3 The clear distance limitations between bars required in Sections 6.1.3.1 and 6.1.3.2 shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

6.1.3.4 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to two in any one bundle. Individual bars in a bundle cut off within the span of a member shall terminate at points at least 40 bar diameters apart.

6.1.3.5 Reinforcement embedded in grout shall have a thickness of grout between the reinforcement and masonry units not less than 7 mm for fine grout or 13 mm for coarse grout.

6.1.4 Protection of reinforcement and metal accessories

6.1.4.1 Reinforcing bars shall have a masonry cover not less than the following:

(a) Masonry face exposed to earth or weather: 50 mm for bars larger than Dia 16; 38 mm for Dia 16 bars or smaller.
(b) Masonry not exposed to earth or weather: 38 mm.

6.1.4.2 Longitudinal wires of joint reinforcement shall be fully embedded in mortar or grout with a minimum cover of 16 mm when exposed to earth or weather and 13 mm when not exposed to earth or weather. Joint reinforcement shall be stainless steel or protected from corrosion by hot-dipped galvanized coating or epoxy coating when used in masonry exposed to earth or weather and in interior walls exposed to a mean relative humidity exceeding 75 percent. All other joint reinforcement shall be mill galvanized, hot-dip galvanized, or stainless steel.

6.1.4.3 Wall ties, sheet-metal anchors, steel plates and bars, and inserts exposed to earth or weather, or exposed to a mean relative humidity exceeding 75 percent shall be stainless steel or protected from corrosion by hot-dip galvanized coating or epoxy coating. Wall ties, anchors, and inserts shall be mill galvanized, hot-dip galvanized, or stainless steel for all other cases. Anchor bolts, steel plates, and bars not exposed to earth, weather, nor exposed to a mean relative humidity exceeding 75 percent, need not be coated.

6.1.5 Standard hooks

Standard hooks shall consist of the following:

(a) 180-degree bend plus a minimum $4d_b$ extension, but not less than 65 mm, at free end of bar;
(b) 90-degree bend plus a minimum $12d_b$ extension at free end of bar; or
(c) for stirrup and tie hooks for a Dia 16 bar and smaller, either a 90-degree or 135-degree bend plus a minimum $6d_b$ extension, but not less than 65 mm, at free end of bar.

6.1.6 Minimum bend diameter for reinforcing bars

The diameter of bend measured on the inside of reinforcing bars, other than for stirrups and ties, shall not be less than values specified in Table 6.1.

6.2—Anchor bolts

Headed and bent-bar anchor bolts shall conform to the provisions of Sections 6.2.1 through 6.2.7.

6.2.1 Placement

Headed and bent-bar anchor bolts shall be embedded in grout. Anchor bolts of 7 mm diameter are permitted to be placed in mortar bed joints that are at least 13 mm in thickness and, for purposes of application of the provisions of Sections 6.2, 8.1.3 and 9.1.6, are permitted to be considered as if they are embedded in grout. Anchor bolts placed in the top of grouted cells and bond beams shall be positioned to maintain a minimum of 7 mm of fine grout between the bolts and the masonry unit or 13 mm of coarse grout between the bolts and the masonry unit. Anchor bolts placed in drilled holes in the face shells of hollow masonry units shall be permitted to contact the masonry unit where the bolt passes through the face shell, but the portion of the bolt that is within the grouted cell shall be positioned to maintain a minimum of 7 mm of fine grout between the head or bent leg of each bolt and the masonry unit or 13 mm of coarse grout between the head or bent leg of each bolt and the masonry unit. The clear distance between parallel anchor bolts shall not be less than the nominal diameter of the anchor bolt, nor less than 25 mm.

6.2.2 Projected area for axial tension

The projected area of headed and bent-bar anchor bolts loaded in axial tension, A_{pt} , shall be determined by 6-1.

$$A_{pt} = \pi l_b^2 \qquad \text{6-1}$$

The portion of projected area overlapping an open cell, or open head joint, or that lies outside the masonry shall be deducted from the value of A_{pt} calculated using 6-1. Where the projected areas of anchor bolts overlap, the value of A_{pt} calculated using 6-1 shall be adjusted so that no portion of masonry is included more than once.

6.2.3 Projected area for shear

The projected area of headed and bent-bar anchor bolts loaded in shear, A_{pv}, shall be determined from 6-2.

$$A_{pv} = \frac{\pi l_{be}^2}{2} \qquad \text{6-2}$$

The portion of projected area overlapping an open cell, or open head joint, or that lies outside the masonry shall be deducted from the value of A_{pv} calculated using 6-2. Where the projected areas of anchor bolts overlap, the value of A_{pv} calculated using 6-2 shall be adjusted so that no portion of masonry is included more than once.

6.2.4 Effective embedment length for headed anchor bolts

The effective embedment length for a headed anchor bolt, l_b, shall be the length of the embedment measured perpendicular from the masonry surface to the compression bearing surface of the anchor head.

6.2.5 Effective embedment length for bent-bar anchor bolts

The effective embedment for a bent-bar anchor bolt, l_b, shall be the length of embedment measured perpendicular from the masonry surface to the compression bearing surface of the bent end, minus one anchor bolt diameter.

6.2.6 Minimum permissible effective embedment length

The minimum permissible effective embedment length for headed and bent-bar anchor bolts shall be the greater of 4 bolt diameters or 50 mm.

6.2.7 Anchor bolt edge distance

Anchor bolt edge distance, l_{be}, shall be measured in the direction of load from the edge of masonry to center of the cross section of anchor bolt.

TABLES OF CHAPTER 6

Table 6.1: Minimum diameters of bend

Bar size and type	Minimum diameter
Dia 10 through Dia 22 (Grade 280)	5 bar diameters
Dia 10 through Dia 25 (Grade 350 or 420)	6 bar diameters
Dia 28, Dia 32, and Dia 36 (Grade 350 or 420)	8 bar diameters

Table 6.2: Physical properties of steel reinforcing wire

Designation	Nominal diameter, mm	Nominal area, mm^2	Nominal mass, kg/m	Area (As, mm^2) per meter Center-to-center spacing, mm								
				50	75	100	150	200	250	300	350	400
WD 4.0	4	12.6	0.099	252	168	126	84	63	50	42	36	32
WD 4.5	4.5	15.9	0.125	318	212	159	106	80	64	53	45	40
WD 5.0	5	19.6	0.154	392	261	196	131	98	78	65	56	49
WD 5.5	5.5	23.8	0.187	476	317	238	159	119	95	79	68	60
WD 6.0	6	28.3	0.222	566	377	283	189	142	113	94	81	71
WD 6.5	6.5	33.2	0.26	664	443	332	221	166	133	111	95	83
WD 7.0	7	38.5	0.302	770	513	385	257	193	154	128	110	96
WD 7.5	7.5	44.2	0.347	884	589	442	295	221	177	147	126	111
WD 8.0	8	50.3		1006	671	503	335	252	201	168	144	126
WD 8.5	8.5	56.7	0.445	1134	756	567	378	284	227	189	162	142
WD 9.0	9	63.6	0.499	1272	848	636	424	318	254	212	182	159
WD 9.5	9.5	70.9	0.556	1418	945	709	473	355	284	236	203	177
WD 10.0	10	78.5	0.617	1570	1047	785	523	393	314	262	224	196
WD 10.5	10.5	86.6	0.68	1732	1155	866	577	433	346	289	247	217
WD 11.0	11	95	0.746	1900	1267	950	633	475	380	317	271	238
WD 11.5	11.5	103.9	0.815	2078	1385	1039	693	520	416	346	297	260
WD 12.0	12	113.1	0.888	2262	1508	1131	754	566	452	377	323	283

Table 6.3: Physical properties of steel reinforcing bars

Bar designation	Nominal diameter, mm	Nominal area, mm^2	Nominal mass, kg/m
Dia 6	6	28	0.222
Dia 8	8	50	0.395
Dia 10	10	79	0.617
Dia 12	12	113	0.888
Dia 14	14	154	1.21
Dia 16	16	201	1.58
Dia 18	18	254	2.00
Dia 20	20	314	2.47
Dia 22	22	380	2.98
Dia 25	25	491	3.85
Dia 28	28	616	4.83
Dia 32	32	804	6.31
Dia 36	36	1018	7.99
Dia 40	40	1257	9.87
Dia 45	45	1590	12.5
Dia 50	50	1963	15.4

CHAPTER 7—SEISMIC DESIGN REQUIREMENTS

7.1—Scope

The seismic design requirements of Chapter 7 shall apply to the design and construction of masonry, except glass unit masonry and masonry veneer.

7.2—General analysis

7.2.1 Element interaction — The interaction of structural and nonstructural elements that affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

7.2.2 Load path — Structural masonry elements that transmit forces resulting from earthquakes to the foundation shall comply with the requirements of Chapter 7.

7.2.3 Anchorage design — Load path connections and minimum anchorage forces shall comply with the requirements of SBC 301.

7.2.4 Drift limits — Under loading combinations that include earthquake, masonry structures shall be designed so the calculated story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from SBC 301.

It shall be permitted to assume that the following shear wall types comply with the story drift limits of SBC 301: empirical, ordinary plain (unreinforced), detailed plain (unreinforced), ordinary reinforced, intermediate reinforced, ordinary plain (unreinforced) AAC masonry shear walls, and detailed plain (unreinforced) AAC masonry shear walls.

7.3—Element classification

Masonry elements shall be classified in accordance with Section 7.3.1 and 7.3.2 as either participating or nonparticipating elements of the seismic-force-resisting system.

7.3.1 Nonparticipating elements — Masonry elements that are not part of the seismic-force-resisting system shall be classified as nonparticipating elements and shall be isolated in their own plane from the seismic force-resisting system except as required for gravity support. Isolation joints and connectors shall be designed to accommodate the design story drift.

7.3.2 Participating elements — Masonry walls that are part of the seismic-force-resisting system shall be classified as participating elements and shall comply with the requirements of Section 7.3.2.1, 7.3.2.2, 7.3.2.3, 7.3.2.4, 7.3.2.5, 7.3.2.6, 7.3.2.7, 7.3.2.8, or 7.3.2.9.

7.3.2.1 Empirical design of masonry shear walls — Empirical design of shear walls shall comply with the requirements of Section A.3.

7.3.2.2 Ordinary plain (unreinforced) masonry shear walls — Design of ordinary plain (unreinforced) masonry shear walls shall comply with the requirements of Section 8.2 or Section 9.2.

7.3.2.3 Detailed plain (unreinforced) masonry shear walls — Design of detailed plain (unreinforced) masonry shear walls shall comply with the requirements of Section 8.2 or Section 9.2, and shall comply with the requirements of Section 7.3.2.3.1.

7.3.2.3.1 Minimum reinforcement requirements — Vertical reinforcement of at least 129 mm^2 in cross-sectional area shall be provided at comers, within 400 mm of each side of openings, within 200 mm of each side of movement joints, within 200 mm of the ends of walls, and at a maximum spacing of 3000 mm on center.

Vertical reinforcement adjacent to openings need not be provided for openings smaller than 400 mm, unless the distributed reinforcement is interrupted by such openings.

Horizontal reinforcement shall consist of at least two longitudinal wires of WD 4.0 joint reinforcement spaced not more than 400 mm on center, or at least 129 mm^2 in cross-sectional area of bond beam reinforcement spaced not more than 3000 mm on center. Horizontal reinforcement shall also be provided: at the bottom and top of wall openings and shall extend at least 625 mm but not less than 40 bar diameters past the opening; continuously at structurally connected roof and floor levels; and within 400 mm of the top of walls.

Horizontal reinforcement adjacent to openings need not be provided for openings smaller than 400 mm, unless the distributed reinforcement is interrupted by such openings.

7.3.2.4 Ordinary reinforced masonry shear walls — Design of ordinary reinforced masonry shear walls shall comply with the requirements of Section 8.3 or Section 9.3, and shall comply with the requirements of Section 7.3.2.3.1.

7.3.2.5 Intermediate reinforced masonry shear walls — Design of intermediate reinforced masonry shear walls shall comply with the requirements of Section 8.3 or Section 9.3. Reinforcement detailing shall also comply with the requirements of Section 7.3.2.3.1, except that the spacing of vertical reinforcement shall not exceed 1200 mm.

7.3.2.6 Special reinforced masonry shear walls — Design of special reinforced masonry shear walls shall comply with the requirements of Section 8.3, Section 9.3, or APPENDIX C. Reinforcement detailing shall also comply with the requirements of Section 7.3.2.3.1 and the following:

(a) The maximum spacing of vertical reinforcement shall be the smallest of one-third the length of the shear wall, one-third the height of the shear wall, and 1200 mm for masonry laid in running bond and 600 mm for masonry not laid in running bond.

(b) The maximum spacing of horizontal reinforcement required to resist in-plane shear shall be uniformly distributed, shall be the smaller of one-third the length of the shear wall and one-third the height of the shear wall, and shall be embedded in grout. The maximum spacing of horizontal reinforcement shall not exceed 1200 mm for masonry laid in running bond and 600 mm for masonry not laid in running bond.

(c) The minimum cross-sectional area of vertical reinforcement shall be one-third of the required shear reinforcement. The sum of the cross-sectional area of horizontal and vertical reinforcement shall be at least 0.002 multiplied by the gross cross-sectional area of the wall, using specified dimensions.

  (1) For masonry laid in running bond, the minimum cross-sectional area of reinforcement in each direction shall be at least 0.0007 multiplied by the gross cross-sectional area of the wall, using specified dimensions.

  (2) For masonry not laid in running bond, the minimum cross-sectional area of vertical reinforcement shall be at least 0.0007 multiplied by the gross cross-sectional area of the wall, using specified dimensions. The minimum cross-sectional area of horizontal reinforcement shall be at least 0.0015 multiplied by the gross cross-sectional area of the wall, using specified dimensions.

(d) Shear reinforcement shall be anchored around vertical reinforcing bars with a standard hook.

(e) Mechanical splices in flexural reinforcement in plastic hinge zones shall develop the specified tensile strength of the spliced bar.

(f) Masonry not laid in running bond shall be fully grouted and shall be constructed of hollow open-end units or two wythes of solid units.

7.3.2.6.1 Shear capacity design

7.3.2.6.1.1 When designing special reinforced masonry shear walls to resist in-plane forces in accordance with Section 9.3, the design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the element, except that the nominal shear strength, V_n, need not exceed 2.5 times required shear strength, V_u.

7.3.2.6.1.2 When designing special reinforced masonry shear walls in accordance with Section 8.3, the shear or diagonal tension stress resulting from in-plane seismic forces shall be increased by a factor of 1.5. The 1.5 multiplier need not be applied to the overturning moment.

7.3.2.7 Ordinary plain (unreinforced) AAC masonry shear walls — Design of ordinary plain (unreinforced) AAC masonry shear walls shall comply with the requirements of Section 11.2 and Section 7.3.2.7.1.

7.3.2.7.1 Anchorage of floor and roof diaphragms in AAC masonry structures — Floor and roof diaphragms in AAC masonry structures shall be anchored to a continuous grouted bond beam reinforced with at least two longitudinal reinforcing bars, having a total cross-sectional area of at least 260 mm^2.

7.3.2.8 Detailed plain (unreinforced) AAC masonry shear walls — Design of detailed plain (unreinforced) AAC masonry shear walls shall comply with the requirements of Section 11.2 and Sections 7.3.2.7.1 and 7.3.2.8.1.

7.3.2.8.1 Minimum reinforcement requirements — Vertical reinforcement of at least 129 mm^2 shall be provided within 600 mm of each side of openings, within 200 mm of movement joints, and within 600 mm of the ends of walls. Vertical reinforcement adjacent to openings need not be provided for openings smaller than 400 mm, unless the distributed reinforcement is interrupted by such openings. Horizontal reinforcement shall be provided at the bottom and top of wall openings and shall extend at least 600 mm but not less than 40 bar diameters past the opening. Horizontal reinforcement adjacent to openings need not be provided for openings smaller than 400 mm, unless the distributed reinforcement is interrupted by such openings.

7.3.2.9 Ordinary reinforced AAC masonry shear walls — Design of ordinary reinforced AAC masonry shear walls shall comply with the requirements of Section 11.3 and Sections 7.3.2.7.1 and 7.3.2.8.1.

7.3.2.9.1 Shear capacity design — The design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the element, except that the nominal shear strength, V_n, need not exceed 2.5 times required shear strength, V_u.

7.3.2.10 Ordinary plain (unreinforced) prestressed masonry shear walls — Design of plain (unreinforced) prestressed masonry shear walls is beyond the scope of the current SBC code.

7.3.2.11 Intermediate reinforced prestressed masonry shear walls — Design of intermediate reinforced prestressed masonry shear walls is beyond the scope of the current SBC code.

7.3.2.12 Special reinforced prestressed masonry shear walls — Design of special reinforced prestressed masonry shear walls is beyond the scope of the current SBC code.

7.4—Seismic Design Category requirements

The design of masonry elements shall comply with the requirements of Sections 7.4.1 through 7.4.4 based on the Seismic Design Category as defined in SBC 301.

7.4.1 Seismic Design Category A requirements — Masonry elements in structures assigned to Seismic Design Category A shall comply with the requirements of Sections 7.1, 7.2, 7.4.1.1, and 7.4.1.2.

7.4.1.1 Design of nonparticipating elements — Nonparticipating masonry elements shall comply with the requirements of Section 7.3.1 and CHAPTER 8, CHAPTER 9, CHAPTER 11, CHAPTER 14, APPENDIX A, or APPENDIX B.

7.4.1.2 Design of participating elements — Participating masonry elements shall be designed to comply with the requirements of CHAPTER 8, CHAPTER 9, CHAPTER 11, CHAPTER 14, APPENDIX A, or APPENDIX B. Masonry shear walls shall be designed to comply with the requirements of Section 7.3.2.1, 7.3.2.2, 7.3.2.3, 7.3.2.4, 7.3.2.5, 7.3.2.6, 7.3.2.7, 7.3.2.8, or 7.3.2.9.

7.4.2 Seismic Design Category B requirements — Masonry elements in structures assigned to Seismic Design Category B shall comply with the requirements of Section 7.4.1 and with the additional requirements of Section 7.4.2.1.

7.4.2.1 Design of participating elements — Participating masonry elements shall be designed to comply with the requirements of CHAPTER 8, CHAPTER 9, CHAPTER 11, or APPENDIX B. Masonry shear walls shall be designed to comply with the requirements of Section 7.3.2.2, 7.3.2.3, 7.3.2.4, 7.3.2.5, 7.3.2.6, 7.3.2.7, 7.3.2.8, or 7.3.2.9.

7.4.3 Seismic Design Category C requirements — Masonry elements in structures assigned to Seismic Design Category C shall comply with the requirements of Section 7.4.2 and with the additional requirements of Section 7.4.3.1 and 7.4.3.2.

7.4.3.1 Design of nonparticipating elements — Nonparticipating masonry elements shall comply with the requirements of Section 7.3.1 and CHAPTER 8, CHAPTER 9, CHAPTER 11, APPENDIX A, or APPENDIX B. Nonparticipating masonry elements, except those constructed of AAC masonry, shall be reinforced in either the horizontal or vertical direction in accordance with the following:

(a) Horizontal reinforcement — Horizontal reinforcement shall consist of at least two longitudinal wires of WD 4.0 bed joint reinforcement spaced not more than 400 mm on center for walls greater than 100 mm in width and at least one longitudinal WD 4.0 wire spaced not more than 400 mm on center for walls not exceeding 100 mm in width or at least one Dia 14 bar spaced not more than 1200 mm on center. Where two longitudinal wires of joint reinforcement are used, the space between these wires shall be the widest that the mortar joint will accommodate. Horizontal reinforcement shall be provided within 400 mm of the top and bottom of these masonry walls.

(b) Vertical reinforcement — Vertical reinforcement shall consist of at least one Dia 14 bar spaced not more than 3000 mm.

Vertical reinforcement shall be located within 400 mm of the ends of masonry walls.

7.4.3.2 Design of participating elements — Participating masonry elements shall be designed to comply with the requirements of Section 8.3, 9.3, 11.3, or APPENDIX B. Masonry shear walls shall be designed to comply with the requirements of Section 7.3.2.4, 7.3.2.5, 7.3.2.6, or 7.3.2.9.

7.4.3.2.1 Connections to masonry columns — Where anchor bolts are used to connect horizontal elements to the tops of columns, anchor bolts shall be placed within lateral ties. Lateral ties shall enclose both the vertical bars in the column and the anchor bolts. There shall be a minimum of two Dia 14 lateral ties provided in the top 125 mm of the column.

7.4.3.2.2 Anchorage of floor and roof diaphragms in AAC masonry structures — Seismic load between floor and roof diaphragms and AAC masonry shear walls shall be transferred through connectors embedded in grout and designed in accordance with Section 4.1.4.

7.4.3.2.3 Material requirements — ASTM C34, structural clay load-bearing wall tiles, shall not be used as part of the seismic-force-resisting system.

7.4.3.2.4 Lateral stiffness — At each story level, at least 80 percent of the lateral stiffness shall be provided by seismic-force-resisting walls. Along each line of lateral resistance at a particular story level, at least 80 percent of the lateral stiffness shall be provided by seismic-force-resisting walls. Where seismic loads are determined based on a seismic response modification factor, R, not greater than 1.5, piers and columns shall be permitted to be used to provide seismic load resistance.

7.4.3.2.5 Design of columns, pilasters, and beams supporting discontinuous elements — Columns and pilasters that are part of the seismic-force resisting system and that support reactions from discontinuous stiff elements shall be provided with transverse reinforcement spaced at no more than one-fourth of the least nominal dimension of the column or pilaster. The minimum transverse reinforcement ratio shall be 0.0015. Beams supporting reactions from discontinuous walls shall be provided with transverse reinforcement spaced at no more than one-half of the nominal depth of the beam. The minimum transverse reinforcement ratio shall be 0.0015.

7.4.4 Seismic Design Category D requirements — Masonry elements in structures assigned to Seismic Design Category D shall comply with the requirements of Section 7.4.3 and with the additional requirements of Sections 7.4.4.1 and 7.4.4.2.

Exception: Design of participating elements of AAC masonry shall comply with the requirements of Section 7.4.3.

7.4.4.1 Design of nonparticipating elements - Nonparticipating masonry elements shall comply with the requirements of CHAPTER 8, CHAPTER 9, CHAPTER 11, or APPENDIX B. Nonparticipating masonry elements, except those constructed of AAC masonry, shall be reinforced in either the horizontal or vertical direction in accordance with the following:

(a) Horizontal reinforcement — Horizontal reinforcement shall comply with Section 7.4.3.1(a).

(b) Vertical reinforcement — Vertical reinforcement shall consist of at least one Dia 14 bar spaced not more than 1200 mm. Vertical reinforcement shall be located within 400 mm of the ends of masonry walls.

7.4.4.2 Design of participating elements — Masonry shear walls shall be designed to comply with the requirements of Section 7.3.2.6 or 7.3.2.9.

7.4.4.2.1 Minimum reinforcement for masonry columns — Lateral ties in masonry columns shall be spaced not more than 200 mm on center and shall be at least 9.5 mm diameter. Lateral ties shall be embedded in grout.

7.4.4.2.2 Material requirements — Fully grouted participating elements shall be designed and specified with Type S or Type M cement-lime mortar, masonry cement mortar, or mortar cement mortar. Partially grouted participating elements shall be designed and specified with Type S or Type M cement-lime mortar or mortar cement mortar

7.4.4.2.3 Lateral tie anchorage — Standard hooks for lateral tie anchorage shall be either a 135-degree standard hook or a 180-degree standard hook.

TABLES OF CHAPTER 7

Table 7.1: Requirements for Masonry Shear Walls Based on Shear Wall Designation[1]

Shear Wall Designation	Design Methods	Reinforcement Requirements	Permitted In
Empirical Design of Masonry Shear Walls	Section A.3	None	SDC A
Ordinary Plain (Unreinforced) Masonry Shear Walls	Section 8.2 or Section 9.2	None	SDC A and B
Detailed Plain (Unreinforced) Masonry Shear Walls	Section 8.2 or Section 9.2	Section 7.3.2.3.1	SDC A and B
Ordinary Reinforced Masonry Shear Walls	Section 8.3 or Section 9.3	Section 7.3.2.3.1	SDC A, B, and C
Intermediate Reinforced Masonry Shear Walls	Section 8.3 or Section 9.3	Section 7.3.2.5	SDC A, B, and C
Special reinforced masonry shear walls	Section 8.3 or Section 9.3	Section 7.3.2.6	SDC A, B, C, and D
Ordinary Plain (Unreinforced) AAC Masonry Shear Walls	Section 11.2	Section 7.3.2.7.1	SDC A and B
Detailed Plain (Unreinforced) AAC Masonry Shear Walls	Section 11.2	Section 7.3.2.8.1	SDC A and B
Ordinary Reinforced AAC masonry shear walls	Section 11.3	Section 7.3.2.9	SDC A, B, C, and D

[1] Section and Chapter references in this table refer to Code Sections and Chapters.

PART 3—ENGINEERED DESIGN METHOD

CHAPTER 8—ALLOWABLE STRESS DESIGN OF MASONRY

8.1—General

8.1.1 Scope

This chapter provides requirements for allowable stress design of masonry. Masonry designed in accordance with this chapter shall comply with the requirements of Part 1, Part 2, Sections 8.1.2 through 8.1.6, and either Section 8.2 or 8.3.

8.1.2 Design strength

Calculated stresses shall not exceed the allowable stress requirements of this Chapter.

8.1.3 Anchor bolts embedded in grout

8.1.3.1 Design requirements — Anchor bolts shall be designed using either the provisions of Section 8.1.3.2 or, for headed and bent-bar anchor bolts, by the provisions of Section 8.1.3.3.

8.1.3.2 Allowable loads determined by test

8.1.3.2.1 Anchor bolts shall be tested in accordance with ASTM E488, except that a minimum of five tests shall be performed. Loading conditions of the test shall be representative of intended use of the anchor bolt.

8.1.3.2.2 Anchor bolt allowable loads used for design shall not exceed 20 percent of the average failure load from the tests.

8.1.3.3 Allowable loads determined by calculation for headed and bent-bar anchor bolts — Allowable loads for headed and bent-bar anchor bolts embedded in grout shall be determined in accordance with the provisions of Sections 8.1.3.3.1 through 8.1.3.3.3.

8.1.3.3.1 Allowable axial tensile load of headed and bent-bar anchor bolts — The allowable axial tensile load of headed anchor bolts shall be calculated using the provisions of Sections 8.1.3.3.1.1. The allowable axial tensile load of bent-bar anchor bolts shall be calculated using the provisions of Section 8.1.3.3.1.2.

8.1.3.3.1.1 Allowable axial tensile load of headed anchor bolts —The allowable axial tensile load, B_a, of headed anchor bolts embedded in grout shall be the smaller of the values determined by Equation 8-1 and Equation 8-2.

$$B_{ab} = 0.104 A_{pt}\sqrt{f'_m} \qquad \text{Equation 8-1}$$

$$B_{as} = 0.6 A_b f_y \qquad \text{Equation 8-2}$$

8.1.3.3.1.2 Allowable axial tensile load of bent-bar anchor bolts — The allowable axial tensile load, B_a, for bent-bar anchor bolts embedded in grout shall be the smallest of the values determined by Equation 8-3,

$$B_{ab} = 0.104 A_{pt}\sqrt{f'_m} \qquad \text{Equation 8-3}$$

$$B_{ap} = 0.6 f'_m e_b d_b + 0.83\pi(l_b + e_b + d_b)d_b \qquad \text{Equation 8-4}$$

$$B_{as} = 0.6 A_b f_y \qquad \text{Equation 8-5}$$

8.1.3.3.2 Allowable shear load of headed and bent-bar anchor bolts — The allowable shear load, B_v, of headed and bent-bar anchor bolts embedded in grout shall be the smallest of the values determined by Equation 8-6, Equation 8-7, Equation 8-8, and Equation 8-9.

$$B_{vb} = 0.104 A_{pv}\sqrt{f'_m} \qquad \text{Equation 8-6}$$

$$B_{vc} = 1072\sqrt[4]{f'_m A_b} \qquad \text{Equation 8-7}$$

$$B_{vpry} = 2.0 B_{ab} = 0.208 A_{pt}\sqrt{f'_m} \qquad \text{Equation 8-8}$$

$$B_{vs} = 0.36 A_b f_y \qquad \text{Equation 8-9}$$

8.1.3.3.3 Combined axial tension and shear — Anchor bolts subjected to axial tension in combination with shear shall satisfy Equation 8-10.

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \le 1 \qquad \text{Equation 8-10}$$

8.1.4 Shear stress in multiwythe masonry elements

8.1.4.1 Design of multiwythe masonry for composite action shall meet the requirements of Section 5.1.4.2 and Section 8.1.4.2.

8.1.4.2 Shear stresses developed at the interfaces between wythes and collar joints or within headers shall not exceed the following:

(a) Mortared collar joints, 48 kPa.
(b) Grouted collar joints, 89 kPa.
(c) headers, $0.108\sqrt{\text{specified unit compressive strength, of header}}$ MPa (over net area of header).

8.1.5 Bearing stress

Bearing stresses on masonry shall not exceed $0.33f'_m$ and shall be calculated over the bearing area, A_{br}, as defined in Section 4.3.4.

8.1.6 Development of reinforcement embedded in grout

8.1.6.1 General — The calculated tension or compression in the reinforcement at each section shall be developed on each side of the section by development length, hook, mechanical device, or combination thereof. Hooks shall not be used to develop bars in compression.

8.1.6.2 Development of wires in tension — The development length of wire shall be determined by Equation 8-11 but shall not be less than 150 mm.

$$l_d = 0.22d_bF_s \qquad \text{Equation 8-11}$$

Development length of epoxy-coated wire shall be taken as 150 percent of the length determined by Equation 8-11.

8.1.6.3 Development of bars in tension or compression — The required development length of reinforcing bars shall be determined by Equation 8-12, but shall not be less than 300 mm.

$$l_d = \frac{1.57d_b^2f_y\gamma}{K\sqrt{f'_m}} \qquad \text{Equation 8-12}$$

K shall not exceed the smallest of the following: the minimum masonry cover, the clear spacing between adjacent reinforcement splices, and $9d_b$.

(a) γ = 1.0 for Dia 10 through Dia 16 bars;
(b) γ = 1.3 for Dia 18 through Dia 22 bars;
(c) γ = 1.5 for Dia 25 through Dia 36 bars.

Development length of epoxy-coated bars shall be taken as 150 percent of the length determined by Equation 8-12.

8.1.6.4 Embedment of flexural reinforcement

8.1.6.4.1 General

8.1.6.4.1.1 Tension reinforcement is permitted to be developed by bending across the neutral axis of the member to be anchored or made continuous with reinforcement on the opposite face of the member.

8.1.6.4.1.2 Critical sections for development of reinforcement in flexural members are at points of maximum steel stress and at points within the span where adjacent reinforcement terminates or is bent.

8.1.6.4.1.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member or $12d_b$, whichever is greater, except at supports of simple spans and at the free end of cantilevers.

8.1.6.4.1.4 Continuing reinforcement shall extend a distance l_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure as required by Section 8.1.6.2 or 8.1.6.3.

8.1.6.4.1.5 Flexural reinforcement shall not be terminated in a tension zone unless one of the following conditions is satisfied:

(a) Shear at the cutoff point does not exceed two-thirds of the allowable shear at the section considered.
(b) Stirrup area in excess of that required for shear is provided along each terminated bar or wire over a distance from the termination point equal to three-fourths the effective depth of the member. Excess stirrup area, A_v, shall not be less than $60b_ws/f_y$. Spacing s shall not exceed $d/(8\beta_b)$.
(c) Continuous reinforcement provides double the area required for flexure at the cutoff point and shear does not exceed three-fourths the allowable shear at the section considered.

8.1.6.4.1.6 Anchorage complying with Section 8.1.6.2 or 8.1.6.3 shall be provided for tension reinforcement in corbels, deep flexural members, variable-depth arches, members where flexural reinforcement is not parallel with the compression face, and in other cases where the stress in flexural reinforcement does not vary linearly through the depth of the section.

8.1.6.4.2 Development of positive moment reinforcement — When a wall or other flexural member is part of the lateral-force-resisting system, at least 25 percent of the positive moment reinforcement shall extend into the support and be anchored to develop F_s in tension.

8.1.6.4.3 Development of negative moment reinforcement

8.1.6.4.3.1 Negative moment reinforcement in a continuous, restrained, or cantilever member shall be anchored in or through the supporting member in accordance with the provisions of Section 8.1.6.1.

8.1.6.4.3.2 At least one-third of the total reinforcement provided for moment at a support shall extend beyond the point of inflection the greater distance of the effective depth of the member or one-sixteenth of the span.

8.1.6.5 Hooks

8.1.6.5.1 Standard hooks in tension shall be considered to develop an equivalent embedment length, l_e, equal to $13d_b$.

8.1.6.5.2 The effect of hooks for bars in compression shall be neglected in design calculations.

8.1.6.6 Development of shear reinforcement

8.1.6.6.1 Bar and wire reinforcement

8.1.6.6.1.1 Shear reinforcement shall extend to a distance d from the extreme compression face and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends for its calculated stress.

8.1.6.6.1.2 The ends of single-leg or U-stirrups shall be anchored by one of the following means:

(a) A standard hook plus an effective embedment of $0.5l_d$. The effective embedment of a stirrup leg shall be taken as the distance between the mid-depth of the member, $d/2$, and the start of the hook (point of tangency).

(b) For bar Dia 16 and MD200 wire and smaller, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of $0.33l_d$. The $0.33l_d$ embedment of a stirrup leg shall be taken as the distance between middepth of member $d/2$, , and start of hook (point of tangency).

8.1.6.6.1.3 Between the anchored ends, each bend in the continuous portion of a transverse U-stirrup shall enclose a longitudinal bar.

8.1.6.6.1.4 Longitudinal bars bent to act as shear reinforcement, where extended into a region of tension, shall be continuous with longitudinal reinforcement and, where extended into a region of compression, shall be developed beyond mid depth of the member, $d/2$.

8.1.6.6.1.5 Pairs of U-stirrups or ties placed to form a closed unit shall be considered properly spliced when length of laps are $1.7l_d$. In grout at least 450 mm deep, such splices with $A_v f_y$ not more than 40,000 N per leg shall be permitted to be considered adequate if legs extend the full available depth of grout.

8.1.6.6.2 Welded wire reinforcement

8.1.6.6.2.1 For each leg of welded wire reinforcement forming simple U-stirrups, there shall be either:

(a) Two longitudinal wires at a 50 mm spacing along the member at the top of the U, or

(b) One longitudinal wire located not more than $d/4$ from the compression face and a second wire closer to the compression face and spaced at least 50 mm from the first wire. The second wire shall be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend at least $8d_b$.

8.1.6.6.2.2 For each end of a single-leg stirrup of plain or deformed welded wire reinforcement, there shall be two longitudinal wires spaced a minimum of 50 mm with the inner wire placed at a distance at least $d/4$ or 50 mm from middepth of member, $d/2$. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

8.1.6.7 Splices of reinforcement — Lap splices, welded splices, or mechanical splices are permitted in accordance with the provisions of this section.

8.1.6.7.1 Lap splices — Lap splices shall not be used in plastic hinge zones of special reinforced masonry shear walls. The length of the plastic hinge zone shall be taken as at least 0.15 times the distance between the point of zero moment and the point of maximum moment.

8.1.6.7.1.1 The minimum length of lap for bars in tension or compression shall be determined by Equation 8-12, but not less than 300 mm.

8.1.6.7.1.2 Where reinforcement consisting of Dia 10 or larger bars is placed transversely within the lap, with at least one bar 200 mm or less from each end of the lap, the minimum length of lap for bars in tension or compression determined by Equation 8-12 shall be permitted to be reduced by multiplying by the confinement factor, ξ, determined in accordance with Equation 8-13. The clear space between the transverse bars and the lapped bars shall not exceed 38 mm and the transverse bars shall be fully developed in grouted masonry. The reduced lap splice length shall not be less than $36d_b$.

$$\xi = 1.0 - \frac{11.60A_{sc}}{d_b^{2.5}} \qquad \text{Equation 8-13}$$

where $\frac{11.60A_{sc}}{d_b^{2.5}} \leq 1.0$

A_{sc} is the area of the transverse bars at each end of the lap splice and shall not be taken greater than $226\ \text{mm}^2$.

8.1.6.7.1.3 Bars spliced by noncontact lap splices shall not be spaced transversely farther apart than one-fifth the required length of lap nor more than 200 mm.

8.1.6.7.2 Welded splices — Welded splices shall have the bars butted and welded to develop in tension at least 125 percent of the specified yield strength of the bar. Welding shall conform to AWS D1.4/D1.4M. Reinforcement to be welded shall conform to ASTM A706, or shall be accompanied by a submittal showing its chemical analysis and carbon equivalent as required by AWS D1.4/D1.4M. Existing reinforcement to be welded shall conform to ASTM A706, or shall be analyzed chemically and its carbon equivalent determined as required by AWS D1.4/D1.4M. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls of masonry.

8.1.6.7.3 Mechanical splices — Mechanical splices shall have the bars connected to develop in tension or compression, as required, at least 125 percent of the specified yield strength of the bar. Mechanical splices shall be classified as Type 1 or Type 2 according to Section 21.1.6.1 of SBC 304. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-wall joint of intermediate or special reinforced masonry shear wall system. Type 2 mechanical splices shall be permitted in any location within a member.

8.1.6.7.4 End-bearing splices

8.1.6.7.4.1 In bars required for compression only, the transmission of compressive stress by bearing of square cut ends held in concentric contact by a suitable device is permitted.

8.1.6.7.4.2 Bar ends shall terminate in flat surfaces within $1\frac{1}{2}$ degree of a right angle to the axis of the bars and shall be fitted within 3 degrees of full bearing after assembly.

8.1.6.7.4.3 End-bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

8.1.6.7.5 Splicing of wires in tension

8.1.6.7.5.1 Lap splices — The minimum length of lap for wires in tension shall be determined by Equation 8-11, but shall not be less than 150 mm.

8.1.6.7.5.2 Welded splices — Welded splices shall have the wires welded to develop at least 125 percent of the specified yield strength of the wire in tension.

8.1.6.7.5.3 Mechanical splices — Mechanical splices shall have the wires connected to develop at least 125 percent of the specified yield strength of the wire in tension.

8.2—Unreinforced masonry

8.2.1 Scope

This section provides requirements for the design of unreinforced masonry as defined in Section 2.2. Design of unreinforced masonry by the allowable stress method shall comply with the requirements of Part 1, Part 2, Section 8.1, and Section 8.2.

8.2.2 Design criteria

Unreinforced masonry members shall be designed in accordance with the principles of engineering mechanics and shall be designed to remain uncracked.

8.2.3 Design assumptions

The following assumptions shall be used in the design of unreinforced masonry members:

(a) Strain in masonry is directly proportional to the distance from the neutral axis.
(b) Flexural tensile stress in masonry is directly proportional to strain.

(c) Flexural compressive stress in combination with axial compressive stress in masonry is directly proportional to strain.
(d) Stresses in reinforcement, if present, are neglected when determining the resistance of masonry to design loads.

8.2.4 Axial compression and flexure

8.2.4.1 Axial and flexural compression — Members subjected to axial compression, flexure, or to combined axial compression and flexure shall be designed to satisfy Equation 8-14 and Equation 8-15.

$$\frac{f_a}{F_a}+\frac{f_b}{F_b}\leq 1 \qquad \text{Equation 8-14}$$

$$P\leq\left(\frac{1}{4}\right)P_e \qquad \text{Equation 8-15}$$

where:

(a) For members having an h/r ratio not greater than 99:

$$F_a=\left(\frac{1}{4}\right)f'_m\times\left[1-\left(\frac{h}{140r}\right)^2\right] \qquad \text{Equation 8-16}$$

(b) For members having an h/r ratio greater than 99:

$$F_a=\left(\frac{1}{4}\right)f'_m\left(\frac{70r}{h}\right)^2 \qquad \text{Equation 8-17}$$

(c)

$$F_b=\left(\frac{1}{3}\right)f'_m \qquad \text{Equation 8-18}$$

(d)

$$P_e=\frac{\pi^2E_mI_n}{h^2}\times\left(1-0.577\frac{e}{r}\right)^3 \qquad \text{Equation 8-19}$$

8.2.4.2 Flexural tension — Allowable tensile stresses for masonry elements subjected to out-of-plane or in-plane bending shall be in accordance with the values in Table 8.1. For grouted masonry not laid in running bond, tension parallel to the bed joints shall be assumed to be resisted only by the minimum cross-sectional area of continuous grout that is parallel to the bed joints.

8.2.5 Axial tension

Axial tension resistance of unreinforced masonry shall be neglected in design.

8.2.6 Shear

8.2.6.1 Shear stresses due to forces acting in the direction considered shall be calculated in accordance with Section 4.3.1 and determined by Equation 8-20.

$$f_v=\frac{VQ}{I_nb} \qquad \text{Equation 8-20}$$

8.2.6.2 In-plane shear stresses shall not exceed any of:

(a) $0.125\sqrt{f'_m}$
(b) 0.827 MPa
(c) For running bond masonry not fully grouted;

$0.255+0.45N_v/A_n$

(d) For masonry not laid in running bond, constructed of open end units, and fully grouted;

$0.255+0.45N_v/A_n$

(e) For running bond masonry fully grouted;

$0.414+0.45N_v/A_n$

(f) For masonry not laid in running bond, constructed of other than open end units, and fully grouted;

103kPa

8.2.6.3 The minimum normalized web area of concrete masonry units, determined in accordance with ASTM C140, shall not be less than 187,500 mm^2/m^2 or the calculated shear stresses in the webs shall not exceed the value given in Section 8.2.6.2(a).

8.3—Reinforced masonry

8.3.1 Scope

This section provides requirements for the design of structures in which reinforcement is used to resist tensile forces in accordance with the principles of engineering mechanics and the contribution of the tensile strength of masonry is neglected, except as provided in Section 8.3.5. Design of reinforced masonry by the allowable stress method shall comply with the requirements of Part 1, Part 2, Section 8.1, and Section 8.3.

8.3.2 Design assumptions

The following assumptions shall be used in the design of reinforced masonry:

(a) Strain compatibility exists between the reinforcement, grout, and masonry.
(b) Strains in reinforcement and masonry are directly proportional to the distances from the neutral axis.
(c) Stress is linearly proportional to the strain.
(d) The compressive resistance of steel reinforcement does not contribute to the axial and flexural strengths unless lateral reinforcement is provided in compliance with the requirements of Section 5.3.1.4.
(e) Stresses remain in the elastic range.
(f) Masonry in tension does not contribute to axial and flexural resistances. Axial and flexural tension stresses are resisted entirely by steel reinforcement.

8.3.3 Steel reinforcement — Allowable stresses

8.3.3.1 Tensile stress in bar reinforcement shall not exceed the following:

(a) Grade 40 or Grade 50 reinforcement: 138 MPa
(b) Grade 60 reinforcement: 220 MPa

8.3.3.2 Tensile stress in wire joint reinforcement shall not exceed 205 MPa.

8.3.3.3 When lateral reinforcement is provided in compliance with the requirements of Section 5.3.1.4, the compressive stress in bar reinforcement shall not exceed the values given in Section 8.3.3.1.

8.3.4 Axial compression and flexure

8.3.4.1 Members subjected to axial compression, flexure, or combined axial compression and flexure shall be designed in compliance with Sections 8.3.4.2 through 8.3.4.4.

8.3.4.2 Allowable forces and stresses

8.3.4.2.1 The compressive force in reinforced masonry due to axial load only shall not exceed that given by Equation 8-21 or Equation 8-22:

(a) For members having an h/r ratio not greater than 99:

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s)\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \text{Equation 8-21}$$

(b) For members having an h/r ratio greater than 99:

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s)\left(\frac{70r}{h}\right)^2 \quad \text{Equation 8-22}$$

8.3.4.2.2 The compressive stress in masonry due to flexure or due to flexure in combination with axial load shall not exceed $0.45f'_m$ provided that the calculated compressive stress due to the axial load component, f_a, does not exceed the allowable stress, F_a, in Section 8.2.4.1.

8.3.4.3 Columns — Design axial loads shall be assumed to act at an eccentricity at least equal to 0.1 multiplied by each side dimension. Each axis shall be considered independently.

8.3.4.4 Walls — Special reinforced masonry shear walls having a shear span ratio, $M/(Vd_v)$, equal to or greater than 1.0 and having an axial load, P, greater than $0.05f'_m A_n$, which are subjected to in-plane forces, shall have a maximum ratio of flexural tensile reinforcement, ρ_{max} not greater than that calculated as follows:

$$\rho_{max} = \frac{nf'_m}{2f_y\left(n+\frac{f_y}{f'_m}\right)} \quad \text{Equation 8-23}$$

The maximum reinforcement ratio does not apply in the out-of-plane direction.

8.3.5 Shear

8.3.5.1 Members shall be designed in accordance with Sections 8.3.5.1.1 through 8.3.5.1.4.

8.3.5.1.1 Calculated shear stress in the masonry shall be determined by the relationship:

$$f_v = \frac{V}{A_{nv}} \quad \text{Equation 8-24}$$

8.3.5.1.2 The calculated shear stress, f_v, shall not exceed the allowable shear stress, F_v, where F_v shall be calculated using Equation 8-25 and shall not be taken greater than the limits given by Section 8.3.5.1.2(a) through (c).

$$F_v = (F_{vm} + F_{vs})\gamma_g \quad \text{Equation 8-25}$$

(a) Where $M/(Vd_v) \leq 0.25$:

$$F_v \leq \left(0.249\sqrt{f'_m}\right)\gamma_g \quad \text{Equation 8-26}$$

(b) Where $M/(Vd_v) \geq 1.0$:

$$F_v \leq \left(0.167\sqrt{f'_m}\right)\gamma_g \qquad \text{Equation 8-27}$$

γ_g = 0.75 for partially grouted shear walls and 1.0 otherwise.

(c) The maximum value of F_v for $M/(Vd_v)$ between 0.25 and 1.0 shall be permitted to be linearly interpolated.

8.3.5.1.3 The allowable shear stress resisted by the masonry, F_{vm} shall be calculated using Equation 8-28 for special reinforced masonry shear walls and using Equation 8-29 for other masonry:

$$F_{vm} = 0.25\frac{P}{A_n} + 0.021 \times \left[\left(4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] \qquad \text{Equation 8-28}$$

$$F_{vm} = 0.25\frac{P}{A_n} + 0.042 \times \left[\left(4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] \qquad \text{Equation 8-29}$$

$M/(Vd_v)$ shall be taken as a positive number and need not be taken greater than 1.0.

8.3.5.1.4 The allowable shear stress resisted by the steel reinforcement, F_{vs} shall be calculated using Equation 8-30:

$$F_{vs} = 0.5\left(\frac{A_v F_s d_v}{A_{nv} S}\right) \qquad \text{Equation 8-30}$$

8.3.5.2 Shear reinforcement shall be provided when f_v exceeds F_{vm}. When shear reinforcement is required, the provisions of Section 8.3.5.2.1 and 8.3.5.2.2 shall apply.

8.3.5.2.1 Shear reinforcement shall be provided parallel to the direction of applied shear force. Spacing of shear reinforcement shall not exceed the lesser of $d/2$ or 1200 mm.

8.3.5.2.2 Reinforcement shall be provided perpendicular to the shear reinforcement and shall be at least equal to one-third. The reinforcement shall be uniformly distributed and shall not exceed a spacing of 2.44 m.

8.3.5.3 In composite masonry walls, shear stresses developed in the planes of interfaces between wythes and filled collar joints or between wythes and headers shall meet the requirements of Section 8.1.4.2.

8.3.5.4 In cantilever beams, the maximum shear shall be used. In non-cantilever beams, the maximum shear shall be used except that sections located within a distance $d/2$ from the face of support shall be designed for the same shear as that calculated at a distance $d/2$ from the face of support when the following conditions are met:

(a) Support reaction, in direction of applied shear force, introduces compression into the end regions of the beam, and

(b) No concentrated load occurs between face of support and a distance $d/2$ from face.

TABLES OF CHAPTER 8

Table 8.1: Allowable flexural tensile stresses for clay and concrete masonry, kPa

Direction of flexural tensile stress and masonry type	Mortar types			
	Portland cement/lime or mortar cement		Masonry cement or air entrained Portland cement/lime	
	M or S	N	M or S	N
Normal to bed joints				
Solid units	366	276	221	138
Hollow units[1]				
Ungrouted	228	172	138	83
Fully grouted	448	434	420	400
Parallel to bed joints in running bond				
Solid units	731	552	441	276
Hollow units				
Ungrouted and partially grouted	455	345	276	172
Fully grouted	731	552	441	276
Parallel to bed joints in masonry not laid in running bond				
Continuous grout section parallel to bed joints	917	917	917	917
Other	0	0	0	0

[1] For partially grouted masonry, allowable stresses shall be determined on the basis of linear interpolation between fully grouted hollow units and ungrouted hollow units based on amount (percentage) of grouting.

CHAPTER 9—STRENGTH DESIGN OF MASONRY

9.1—General

9.1.1 Scope

This Chapter provides minimum requirements for strength design of masonry. Masonry design by the strength design method shall comply with the requirements of Part 1, Part 2, Sections 9.1.2 through 9.1.9, and either Section 9.2 or 9.3.

9.1.2 Required strength

Required strength shall be determined in accordance with the strength design load combinations of SBC 301. Members subject to compressive axial load shall be designed for the factored moment accompanying the factored axial load. The factored moment, M_u, shall include the moment induced by relative lateral displacement

9.1.3 Design strength

Masonry members shall be proportioned so that the design strength equals or exceeds the required strength. Design strength is the nominal strength multiplied by the strength-reduction factor, ϕ, as specified in Section 9.1.4.

9.1.4 Strength-reduction factors

9.1.4.1 Anchor bolts — For cases where the nominal strength of an anchor bolt is controlled by masonry breakout, by masonry crushing, or by anchor bolt pryout, ϕ shall be taken as 0.50. For cases where the nominal strength of an anchor bolt is controlled by anchor bolt steel, ϕ shall be taken as 0.90. For cases where the nominal strength of an anchor bolt is controlled by anchor pullout, ϕ shall be taken as 0.65.

9.1.4.2 Bearing — For cases involving bearing on masonry, ϕ shall be taken as 0.60.

9.1.4.3 Combinations of flexure and axial load in unreinforced masonry — The value of ϕ shall be taken as 0.60 for unreinforced masonry subjected to flexure, axial load, or combinations thereof.

9.1.4.4 Combinations of flexure and axial load in reinforced masonry — The value of ϕ shall be taken as 0.90 for reinforced masonry subjected to flexure, axial load, or combinations thereof

9.1.4.5 Shear — The value of ϕ shall be taken as 0.80 for masonry subjected to shear.

9.1.5 Deformation requirements

9.1.5.1 Deflection of unreinforced (plain) masonry — Deflection calculations for unreinforced (plain) masonry members shall be based on uncracked section properties.

9.1.5.2 Deflection of reinforced masonry - Deflection calculations for reinforced masonry members shall consider the effects of cracking and reinforcement on member stiffness. The flexural and shear stiffness properties assumed for deflection calculations shall not exceed one-half of the gross section properties, unless a cracked-section analysis is performed.

9.1.6 Anchor bolts embedded in grout

Anchorage assemblies connecting masonry elements that are part of the seismic force-resisting system to diaphragms and chords shall be designed so that the strength of the anchor is governed by steel tensile or shear yielding. Alternatively, the anchorage assembly is permitted to be designed so that it is governed by masonry breakout or anchor pullout provided that the anchorage assembly is designed to resist not less than 2 times the factored forces transmitted by the assembly.

9.1.6.1 Design requirements — Anchor bolts shall be designed using either the provisions of 9.1.6.2 or, for headed and bent-bar anchor bolts, by the provisions of Section 9.1.6.3.

9.1.6.2 Nominal strengths determined by test

9.1.6.2.1 Anchor bolts shall be tested in accordance with ASTM E488, except that a minimum of five tests shall be performed. Loading conditions of the test shall be representative of intended use of the anchor bolt.

9.1.6.2.2 Anchor bolt nominal strengths used for design shall not exceed 65 percent of the average failure load from the tests.

9.1.6.3 Nominal strengths determined by calculation for headed and bent-bar anchor bolts — Nominal strengths of headed and bent-bar anchor bolts embedded in grout shall be determined in accordance with the provisions of Sections 9.1.6.3.1 through 9.1.6.3.3.

9.1.6.3.1 Nominal tensile strength of headed and bent-bar anchor bolts — The nominal axial tensile strength of headed anchor bolts shall be calculated using the provisions of Sections 9.1.6.3.1.1. The nominal axial tensile strength of bent-bar anchor bolts shall be calculated using the provisions of Section 9.1.6.3.1.2.

9.1.6.3.1.1 Axial tensile strength of headed anchor bolts — The nominal axial tensile strength, Ban of headed anchor bolts embedded in grout shall be determined by Equation 9-1 (nominal axial tensile strength governed by masonry breakout) or Equation 9-2 (nominal axial tensile strength governed by steel yielding). The design axial tensile strength, ϕB_{an} shall be the smaller of the values obtained from Equation 9-1 and Equation 9-2 multiplied by the applicable ϕ value.

$$B_{anb} = 0.332 A_{pt}\sqrt{f'_m} \qquad \text{Equation 9-1}$$

$$B_{ans} = A_b f_y \qquad \text{Equation 9-2}$$

9.1.6.3.1.2 Axial tensile strength of bent-bar anchor bolts – The nominal axial tensile strength, B_{an}, for bent-bar anchor bolts embedded in grout shall be determined by Equation 9-3 (nominal axial tensile strength governed by masonry breakout), Equation 9-4 (nominal axial tensile strength governed by anchor bolt pullout), or Equation 9-5 (nominal axial tensile strength governed by steel yielding). The design axial tensile strength, ϕB_{an}, shall be the smallest of the values obtained from Equation 9-3, Equation 9-4 and Equation 9-5 multiplied by the applicable ϕ value.

$$B_{anb} = 0.332 A_{pt}\sqrt{f'_m} \qquad \text{Equation 9-3}$$

$$B_{anp} = 1.5 f'_m e_b d_b + 2.07\pi(l_b + e_b + d_b) d_b \qquad \text{Equation 9-4}$$

$$B_{ans} = A_b f_y \qquad \text{Equation 9-5}$$

9.1.6.3.2 Shear strength of headed and bent-bar anchor bolts — The nominal shear strength, B_{vn}, of headed and bent-bar anchor bolts shall be determined by Equation 9-6 (nominal shear strength governed by masonry breakout), Equation 9-7 (nominal shear strength governed by masonry crushing), Equation 9-8 (nominal shear strength governed by anchor bolt pryout) or Equation 9-9 (nominal shear strength governed by steel yielding). The design shear strength ϕB_{vn} shall be the smallest of the values obtained from Equation 9-6, Equation 9-7, Equation 9-8 and Equation 9-9 multiplied by the applicable ϕ value.

$$B_{vnb} = 0.332 A_{pt}\sqrt{f'_m} \qquad \text{Equation 9-6}$$

$$B_{vnc} = 3216\sqrt[4]{f'_m A_b} \qquad \text{Equation 9-7}$$

$$B_{vnpry} = 2.0 B_{anb} = 0.664 A_{pt}\sqrt{f'_m} \qquad \text{Equation 9-8}$$

$$B_{vns} = 0.6 A_b f_y \qquad \text{Equation 9-9}$$

9.1.6.3.3 Combined axial tension and shear — Anchor bolts subjected to axial tension in combination with shear shall satisfy Equation 9-10.

$$\frac{b_{af}}{\emptyset B_{an}} + \frac{b_{vf}}{\emptyset B_{vn}} \leq 1 \qquad \text{Equation 9-10}$$

9.1.7 Shear strength in multiwythe masonry elements

9.1.7.1 Design of multiwythe masonry for composite action shall meet the requirements of Sections 5.1.4.2 and 9.1.7.2.

9.1.7.2 The nominal shear strength at the interfaces between wythes and collar joints or within headers shall be determined so that shear stresses shall not exceed the following:

(a) Mortared collar joints, 96 kPa.
(b) Grouted collar joints, 179 kPa.
(c) headers, $0.108\sqrt{\text{specified unit compressive strength, of header}}$ MPa (over net area of header).

9.1.8 Nominal bearing strength

The nominal bearing strength of masonry shall be calculated as $0.8 f'_m$ multiplied by the bearing area, A_{br}, as defined in Section 4.3.4.

9.1.9 Material properties

9.1.9.1 Compressive strength

9.1.9.1.1 Masonry compressive strength — The specified compressive strength of masonry, f'_m shall equal or exceed 10 MPa. The value of f'_m used to determine nominal strength values in this chapter shall not exceed 27 MPa for concrete masonry and shall not exceed 41 MPa for clay masonry.

9.1.9.1.2 Grout compressive strength — For concrete masonry, the specified compressive strength of grout, f'_g shall equal or exceed the specified compressive strength of masonry, f'_m but shall not exceed 35 MPa. For clay masonry, the specified compressive strength of grout, f'_g, shall not exceed 41 MPa.

9.1.9.2 Masonry modulus of rupture — The modulus of rupture, f_r, for masonry elements subjected to out-of-plane or in-plane bending shall be in accordance with the values in Table 9.1. For grouted masonry not laid in running bond, tension parallel to the bed joints shall be assumed to be resisted only by the minimum cross-sectional area of continuous grout that is parallel to the bed joints.

9.1.9.3 Reinforcement strengths

9.1.9.3.1 Reinforcement for in-plane flexural tension and flexural tension perpendicular to bed joints — Masonry design shall be based on a reinforcement strength equal to the specified yield strength of reinforcement, f_y, which shall not exceed 414 MPa. The actual yield strength shall not exceed 1.3 multiplied by the specified yield strength.

9.1.9.3.2 Reinforcement for in-plane shear and flexural tension parallel to bed joints — Masonry design shall be based on a specified yield strength, f_y, which shall not exceed 414 MPa for reinforcing bars and which shall not exceed 586 MPa for reinforcing wire.

9.2—Unreinforced (plain) masonry

9.2.1 Scope

Design of unreinforced masonry by the strength design method shall comply with the requirements of Part 1, Part 2, Section 9.1, and Section 9.2.

9.2.2 Design criteria

Unreinforced masonry members shall be designed in accordance with the principles of engineering mechanics and shall be designed to remain uncracked.

9.2.3 Design assumptions

The following assumptions shall be used in the design of unreinforced masonry members:

(a) Strain in masonry shall be directly proportional to the distance from the neutral axis.

(b) Flexural tension in masonry shall be assumed to be directly proportional to strain.

(c) Flexural compressive stress in combination with axial compressive stress in masonry shall be assumed to be directly proportional to strain.

(d) Stresses in the reinforcement are not accounted for in determining the resistance to design loads

9.2.4 Nominal flexural and axial strength

9.2.4.1 Nominal strength — The nominal strength of unreinforced (plain) masonry cross-sections for combined flexure and axial loads shall be determined so that:

(a) The compressive stress does not exceed $0.80f'_m$.

(b) The tensile stress does not exceed the modulus of rupture determined from Section 9.1.9.2.

9.2.4.2 Nominal axial strength — The nominal axial strength, P_n, shall not be taken greater than the following:

(a) For members having an h/r ratio not greater than 99:

$$P_n = 0.80\left\{\frac{A_n f'_m}{1.25} \times \left[1-\left(\frac{h}{140r}\right)^2\right]\right\} \quad \text{Equation 9-11}$$

(b) For members having an h/r ratio greater than 99:

$$P_n = 0.80 \times \left[0.80 A_n f'_m \left(\frac{70r}{h}\right)^2\right] \quad \text{Equation 9-12}$$

9.2.4.3 P-Delta effects

9.2.4.3.1 Members shall be designed for the factored axial load, P_u, and the moment magnified for the effects of member curvature, M_u.

9.2.4.3.2 The magnified moment, M_u, shall be determined either by a second-order analysis, or by a first-order analysis and Equation 9-13 and Equation 9-14.

$$M_u = \psi M_{u,0} \quad \text{Equation 9-13}$$

$$\psi = \frac{1}{1 - \frac{P_u}{A_n f'_m \left(\frac{70r}{h}\right)^2}} \qquad \text{Equation 9-14}$$

9.2.4.3.3 A value of $\psi = 1$ shall be permitted for members in which $h/r \leq 45$.

9.2.4.3.4 A value of $\psi = 1$ shall be permitted for members in which $45 < h/r \leq 60$, provided that the nominal strength defined in Section 9.2.4.1 is reduced by 10 percent.

9.2.5 Axial tension — axial tension resistance of unreinforced masonry shall be neglected in design.

9.2.6 Nominal shear strength

9.2.6.1 Nominal shear strength, V_n, shall be the smallest of (a), (b) and the applicable condition of (c) through (f):

(a) $0.316A_{nv}\sqrt{f'_m}$
(b) $2.07\ A_{nv}$
(c) For running bond masonry not fully grouted;

$0.386A_{nv} + 0.45N_u$

(d) For masonry not laid in running bond, constructed of open end units, and fully grouted;

$0.386A_{nv} + 0.45N_u$

(e) For running bond masonry fully grouted;

$0.620A_{nv} + 0.45N_u$

(f) For masonry not laid in running bond, constructed of other than open end units, and fully grouted;

$0.159A_{nv}$

9.2.6.2 The minimum normalized web area of concrete masonry units, determined in accordance with ASTM C140, shall not be less than 187,500 mm^2/m^2 or the nominal shear strength of the web shall not exceed $0.316A_{nv}\sqrt{f'_m}I_n b/Q$

9.3—Reinforced masonry

9.3.1 Scope

This section provides requirements for the design of structures in which reinforcement is used to resist tensile forces in accordance with the principles of engineering mechanics and the contribution of the tensile resistance of the masonry is neglected except as provided in Section 9.3.4.1.2. Design of reinforced masonry by the strength design method shall comply with the requirements of Part 1, Part 2, Section 9.19.1, and Section 9.3.

9.3.2 Design assumptions

The following assumptions shall be used in the design of reinforced masonry:

(a) Strain compatibility exists between the reinforcement, grout, and masonry.
(b) The nominal strength of reinforced masonry cross-sections for combined flexure and axial load is based on applicable conditions of equilibrium.
(c) The maximum usable strain, ε_{mu}, at the extreme masonry compression fiber is 0.0035 for clay masonry and 0.0025 for concrete masonry.
(d) Strains in reinforcement and masonry are directly proportional to the distance from the neutral axis.
(e) Compression and tension stress in reinforcement is E_s multiplied by the steel strain, but not greater than f_y. Except as permitted in Section 9.3.3.5.1 (e) for determination of maximum area of flexural reinforcement, the compressive stress of steel reinforcement does not contribute to the axial and flexural resistance unless lateral restraining reinforcement is provided in compliance with the requirements of Section 5.3.1.4.
(f) Masonry in tension does not contribute to axial and flexural strengths. Axial and flexural tension stresses are resisted entirely by steel reinforcement.
(g) The relationship between masonry compressive stress and masonry strain is defined by the following:

Masonry stress of $0.80f'_m$ is uniformly distributed over an equivalent compression stress block bounded by edges of the cross section and a straight line located parallel to the neutral axis and located at a distance $a = 0.80c$ from the fiber of maximum compressive strain. The distance c from the fiber of maximum strain to the neutral axis shall be measured perpendicular to the neutral axis.

9.3.3 Reinforcement requirements and details

9.3.3.1 Reinforcement size limitations

(a) Reinforcing bars used in masonry shall not be larger than Dia 28. The nominal bar diameter shall not exceed one-eighth of the nominal member thickness and shall not exceed one-quarter of the least clear dimension of the cell, course, or collar joint in which the bar is placed. The area of reinforcing bars placed in a cell or in a course of hollow unit construction shall not exceed 4 percent of the cell area

(b) Joint reinforcement longitudinal wire used in masonry as shear reinforcement shall be at least 5 mm diameter.

9.3.3.2 Standard hooks — Standard hooks in tension shall be considered to develop an equivalent embedment length, le, as determined by Equation 9-15:

$$l_e = 13d_b \qquad \text{Equation 9-15}$$

9.3.3.3 Development — The required tension or compression reinforcement shall be developed in accordance with the following provisions:

The required development length of reinforcement shall be determined by Equation 9-16, but shall not be less than 300 mm.

$$l_d = \frac{1.57d_b^2 f_y \gamma}{k\sqrt{f'_m}} \qquad \text{Equation 9-16}$$

K shall not exceed the smallest of the following: the minimum masonry cover, the clear spacing between adjacent reinforcement splices, and $9d_b$.

(a) γ = 1.0 for Dia 10 through Dia 16 bars;
(b) γ = 1.3 for Dia 18 through Dia 22 bars;
(c) γ = 1.5 for Dia 25 through Dia 28 bars.

Development length of epoxy-coated reinforcing bars shall be taken as 150 percent of the length determined by Equation 9-16.

9.3.3.3.1 Reinforcement spliced by noncontact lap splices shall not be spaced farther apart than one-fifth the required length of lap nor more than 200 mm.

9.3.3.3.2 Shear reinforcement shall extend the depth of the member less cover distances.

9.3.3.3.2.1 Except at wall intersections, the end of a horizontal reinforcing bar needed to satisfy shear strength requirements of Section 9.3.4.1.2 shall be bent around the edge vertical reinforcing bar with a 180-degree hook. The ends of single-leg or U-stirrups shall be anchored by one of the following means:

(a) A standard hook plus an effective embedment of $l_d/2$. The effective embedment of a stirrup leg shall be taken as the distance between the mid-depth of the member, $d/2$, and the start of the hook (point of tangency).

(b) For Dia 16 bars and smaller, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of $l_d/3$. The $l_d/3$ embedment of a stirrup leg shall be taken as the distance between mid-depth of the member, $d/2$, and the start of the hook (point of tangency).

(c) Between the anchored ends, each bend in the continuous portion of a transverse U-stirrup shall enclose a longitudinal bar.

9.3.3.3.2.2 At wall intersections, horizontal reinforcing bars needed to satisfy shear strength requirements of Section 9.3.4.1.2 shall be bent around the edge vertical reinforcing bar with a 90-degree standard hook and shall extend horizontally into the intersecting wall a minimum distance at least equal to the development length.

9.3.3.3.2.3 Joint reinforcement used as shear reinforcement and needed to satisfy the shear strength requirements of Section 9.3.4.1.2 shall be anchored around the edge reinforcing bar in the edge cell, either by bar placement between adjacent cross-wires or with a 90-degree bend in longitudinal wires bent around the edge cell and with at least 75-mm bend extensions in mortar or grout.

9.3.3.3.3 Development of wires in tension — the development length of wire shall be determined by Equation 9-17, but shall not be less than 150 mm.

$$l_d = 48d_b \qquad \text{Equation 9-17}$$

Development length of epoxy-coated wire shall be taken as 150 percent of the length determined by Equation 9-17.

9.3.3.4 Splices — Reinforcement splices shall comply with one of the following:

(a) The minimum length of lap for bars shall be 300 mm or the development length determined by Equation 9-16, whichever is greater.

(b) Where reinforcement consisting of M10 or larger bars is placed within the lap, with at least one bar 200 mm or less from each end of the lap, the minimum length of lap for bars in tension or compression determined by Equation 9-16 shall be permitted to be reduced by multiplying the confinement reinforcement factor, ξ. The clear space between the transverse bars and the lapped bars shall not exceed 38 mm and the transverse bars shall be fully developed in grouted masonry. The reduced lap splice length shall not be less than $36d_b$.

$$\xi = 1.0 - \frac{11.60A_{sc}}{d_b^{2.5}} \qquad \text{Equation 9-18}$$

Where: $\frac{11.60A_{sc}}{d_b^{2.5}} \leq 1.0$

A_{sc} is the area of the transverse bars at each end of the lap splice and shall not be taken greater than 226 mm^2.

(c) A welded splice shall be capable of developing in tension at least 125 percent of the specified yield strength, f_y, of the bar. Welded splices shall only be permitted for ASTM A706 steel reinforcement. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls of masonry.

(d) Mechanical splices shall be classified as Type 1 or Type 2 according to Section 21.1.6.1 of SBC 304. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-column joint of intermediate or special reinforced masonry shear walls. Type 2 mechanical splices are permitted in any location within a member.

(e) Where joint reinforcement is used as shear reinforcement, the splice length of the longitudinal wires shall be a minimum of $48d_b$.

9.3.3.4.1 Lap splices shall not be used in plastic hinge zones of special reinforced masonry shear walls. The length of the plastic hinge zone shall be taken as at least 0.15 times the distance between the point of zero moment and the point of maximum moment.

9.3.3.5 Maximum area of flexural tensile reinforcement

9.3.3.5.1 For masonry members where $M_u/(V_u d_v) \geq 1$, the cross-sectional area of flexural tensile reinforcement shall not exceed the area required to maintain axial equilibrium under the following conditions:

(a) A strain gradient shall be assumed, corresponding to a strain in the extreme tensile reinforcement equal to 1.5 multiplied by the yield strain and a maximum strain in the masonry as given by Section 9.3.2(c).

(b) The design assumptions of Section 9.3.2 shall apply.

(c) The stress in the tension reinforcement shall be taken as the product of the modulus of elasticity of the steel and the strain in the reinforcement, and need not be taken greater than f_y.

(d) Axial forces shall be taken from the loading combination given by $D + 0.75L + 0.525Q_E$.

(e) The effect of compression reinforcement, with or without lateral restraining reinforcement, shall be permitted to be included for purposes of calculating maximum flexural tensile reinforcement.

9.3.3.5.2 For intermediate Reinforced masonry shear walls subject to in-plane loads where $M_u/(V_u d_v) \geq 1$, a strain gradient corresponding to a strain in the extreme tensile reinforcement equal to 3 multiplied by the yield strain and a maximum strain in the masonry as given by Section 9.3.2(c) shall be used. For intermediate reinforced masonry shear walls subject to out-of-plane loads, the provisions of Section 9.3.3.5.1 shall apply.

9.3.3.5.3 For special reinforced masonry shear walls subject to in-plane loads where $M_u/(V_u d_v) \geq 1$, a strain gradient corresponding to a strain in the extreme tensile reinforcement equal to 4 multiplied by the yield strain and a maximum strain in the masonry as given by Section 9.3.2(c) shall be used. For special reinforced masonry shear walls subject to out-of-plane loads, the provisions of Section 9.3.3.5.1 shall apply.

9.3.3.5.4 For masonry members where $M_u/(V_u d_v) \leq 1$ and when designed using $R \leq 1.5$, there is no upper limit to the maximum flexural tensile reinforcement. For masonry members where $M_u/(V_u d_v) \leq 1$ and when designed using $R \geq 1.5$, the provisions of Section 9.3.3.5.1 shall apply.

9.3.3.6 Bundling of reinforcing bars — Reinforcing bars shall not be bundled.

9.3.3.7 Joint reinforcement used as shear reinforcement — Joint reinforcement used as shear reinforcement shall consist of at least two 5 mm diameter longitudinal wires located within a bed joint and placed over the masonry unit face shells. The maximum spacing of joint reinforcement used as shear reinforcement shall not exceed 400 mm for Seismic Design Categories (SDC) A and B and shall not exceed 200 mm in partially grouted walls for SDC C and D. Joint reinforcement used as shear reinforcement in fully grouted walls for SDC C and D shall consist of four 5 mm diameter longitudinal wires at a spacing not to exceed 200 mm.

9.3.4 Design of beams, piers, and columns

Member design forces shall be based on an analysis that considers the relative stiffness of structural members. The calculation of lateral stiffness shall include the contribution of all beams, piers, and columns. The effects of cracking on member stiffness shall be considered.

9.3.4.1 Nominal strength

9.3.4.1.1 Nominal axial and flexural strength — The nominal axial strength, P_n, and the nominal flexural strength, M_n, of a cross section shall be determined in accordance with the design assumptions of Section 9.3.2 and the provisions of this Section. The nominal flexural strength at any section along a member shall not be less than one-fourth of the maximum nominal flexural strength at the critical section.

The nominal axial compressive strength shall not exceed Equation 9-19 or Equation 9-20, as appropriate.

(a) For members having an h/r ratio not greater than 99:

$$P_n = 0.80[0.80f'_m(A_n - A_{st}) + f_y A_{st}]\left[1 - \left(\frac{h}{140r}\right)^2\right] \quad \text{Equation 9-19}$$

(b) For members having an h/r ratio greater than 99:

$$P_n = 0.80[0.80f'_m(A_n - A_{st}) + f_y A_{st}]\left(\frac{70r}{h}\right)^2 \quad \text{Equation 9-20}$$

9.3.4.1.2 Nominal shear strength Nominal shear strength, V_n, shall be calculated using Equation 9-21, and shall not be taken greater than the limits given by 9.3.4.1.2 (a) through (c).

$$V_n = (V_{nm} + V_{ns})\gamma_g \quad \text{Equation 9-21}$$

(a) Where $M_u/(V_u d_v) \le 0.25$:

$$V_n \le \left(0.498A_{nv}\sqrt{f'_m}\right)\gamma_g \quad \text{Equation 9-22}$$

(b) Where $M_u/(V_u d_v) \ge 1.0$

$$V_n \le (0.332A_{nv}\sqrt{f'_m})\gamma_g \quad \text{Equation 9-23}$$

$\gamma_g = 0.75$ for partially grouted shear walls and 1.0 otherwise.

(c) The maximum value of V_n for $M_u/(V_n d_v)$ between 0.25 and 1.0 shall be permitted to be linearly interpolated.

9.3.4.1.2.1 Nominal masonry shear strength — Shear strength provided by the masonry, V_{nm}, shall be calculated using Equation 9-24:

$$V_{nm} = 0.083 \times \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] \times A_{nv}\sqrt{f'_m} + 0.25P_u \quad \text{Equation 9-24}$$

$M_u/(V_u d_v)$ shall be taken as a positive number and need not be taken greater than 1.0

9.3.4.1.2.2 Nominal shear strength provided by reinforcement — Nominal shear strength provided by shear reinforcement, V_{ns}, shall be calculated as follows:

$$V_{ns} = 0.5\left(\frac{A_v}{s}\right)f_y d_v \quad \text{Equation 9-25}$$

9.3.4.2 Beams — Design of beams shall meet the requirements of Section 5.2 and the additional requirements of Sections 9.3.4.2.1 through 9.3.4.2.4.

9.3.4.2.1 The factored axial compressive force on a beam shall not exceed $0.05A_n f'_m$

9.3.4.2.2 Longitudinal reinforcement

9.3.4.2.2.1 The variation in longitudinal reinforcing bars in a beam shall not be greater than one bar size. Not more than two bar sizes shall be used in a beam.

9.3.4.2.2.2 The nominal flexural strength of a beam shall not be less than 1.3 multiplied by the nominal cracking moment of the beam, M_{cr}, the modulus of rupture, f_r, for this calculation shall be determined in accordance with Section 9.1.9.2.

9.3.4.2.2.3 The requirements of Section 9.3.4.2.2.2 need not be applied if at every section the area of tensile reinforcement provided is at least one-third greater than that required by analysis.

9.3.4.2.3 Transverse reinforcement — Transverse reinforcement shall be provided where V_u exceeds ϕV_{nm}. The factored shear, V_u, shall include the effects of lateral load. When transverse reinforcement is required, the following provisions shall apply:

(a) Transverse reinforcement shall be a single bar with a 180-degree hook at each end.
(b) Transverse reinforcement shall be hooked around the longitudinal reinforcement.
(c) The minimum area of transverse reinforcement shall be $0.0007bd_v$.
(d) The first transverse bar shall not be located more than one-fourth of the beam depth, d_v, from the end of the beam.
(e) The maximum spacing shall not exceed one-half the depth of the beam nor 1200 mm.

9.3.4.2.4 Construction — Beams shall be fully grouted.

9.3.4.2.5 Coupling beams — Structural members that provide coupling between shear walls shall be designed to reach their moment or shear nominal strength before either shear wall reaches its moment or shear nominal strength. Analysis of coupled shear walls shall comply with accepted principles of mechanics.

The design shear strength, ϕV_n, of the coupling beams shall satisfy the following criterion:

$$\phi Vn \geq \frac{1.25(M_1 + M_2)}{L_c} + 1.4V_g \qquad \text{Equation 9-26}$$

Where M_1 and M_2 are the normal moment strength at the ends of the beam, L_c is the length of the beam between the shear walls and V_g is the unfactored shear force due to gravity loads.

The calculation of the nominal flexural moment shall include the reinforcement in reinforced concrete roof and floor systems. The width of the reinforced concrete used for calculations of reinforcement shall be six times the floor or roof slab thickness.

9.3.4.2.6 Deep flexural member detailing — Flexural members with overall-depth-to-clear-span ratio greater than 2/5 for continuous spans or 4/5 for simple spans shall be detailed in accordance with this section.

9.3.4.2.6.1 Minimum flexural tension reinforcement shall conform to Section 9.3.4.3.2.

9.3.4.2.6.2 Uniformly distributed horizontal and vertical reinforcement shall be provided throughout the length and depth of deep flexural members such that the reinforcement ratios in both directions are at least 0.001. Distributed flexural reinforcement is to be included in the determination of the actual reinforcement ratios.

9.3.4.3 Piers

9.3.4.3.1 The factored axial compression force on piers shall not exceed $0.3A_n f'_m$

9.3.4.3.2 Longitudinal reinforcement — a pier subjected to in-plane stress reversals shall be reinforced symmetrically about the neutral axis of the pier. Longitudinal reinforcement of piers shall comply with the following:

(a) At least one bar shall be provided in each end cell.
(b) The minimum area of longitudinal reinforcement shall be 0.0007bd.

9.3.4.3.3 Dimensional limits — Dimensions shall be in accordance with the following:

(a) The nominal thickness of a pier shall not exceed 400 mm.
(b) The distance between lateral supports of a pier shall not exceed 25 multiplied by the nominal thickness of a pier except as provided for in Section 9.3.4.3.3(c).
(c) When the distance between lateral supports of a pier exceeds 25 multiplied by the nominal thickness of the pier, design shall be based on the provisions of Section 9.3.5.
(d) The nominal length of a pier shall not be less than three multiplied by its nominal thickness nor greater than six multiplied by its nominal thickness. The clear height of a pier shall not exceed five multiplied by its nominal length.

Exception: When the factored axial force at the location of maximum moment is less than $0.05f'_m A_g$, the length of a pier shall be permitted to be equal to the thickness of the pier.

9.3.5 Wall design for out-of-plane loads

9.3.5.1 Scope — The requirements of Section 9.3.5 shall apply to the design of walls for out-of-plane loads.

9.3.5.2 Nominal axial and flexural strength the nominal axial strength, P_n, and the nominal flexural strength, M_n, of a cross-section shall be determined in accordance with the design assumptions of Section 9.3.2. The nominal axial compressive strength shall not exceed that determined by Equation 9-19 or Equation 9-20, as appropriate.

9.3.5.3 Nominal shear strength — The nominal shear strength shall be determined by Section 9.3.4.1.2.

9.3.5.4 P-Delta effects

9.3.5.4.1 Members shall be designed for the factored axial load, P_u, and the moment magnified for the effects of member curvature, M_u. The magnified moment shall be determined either by Section 9.3.5.4.2 or Section 9.3.5.4.3.

9.3.5.4.2 Moment and deflection calculations in this section are based on simple support conditions top and bottom. For other support and fixity conditions, moments and deflections shall be calculated using established principles of mechanics.

The procedures set forth in this Section shall be used when the factored axial load stress at the location of maximum moment satisfies the requirement calculated by Equation 9-27.

$$\left(\frac{P_u}{A_g}\right) \le 0.20 f'_m \qquad \text{Equation 9-27}$$

When the ratio of effective height to nominal thickness, h/t, exceeds 30, the factored axial stress shall not exceed $0.05 f'_m$.

A nominal thickness of 100 mm is permitted where load-bearing reinforced hollow clay unit masonry walls satisfy all of the following conditions.

(1) The maximum unsupported height-to-thickness or length-to-thickness ratios do not exceed 27.

(2) The net area unit strength exceeds 55 MPa.

(3) Units are laid in running bond.

(4) Bar sizes do not exceed 12 mm.

(5) There are no more than two bars or one splice in a cell.

(6) Joints are not raked.

Factored moment and axial force shall be determined at the mid height of the wall and shall be used for design. The factored moment, M_u, at the midheight of the wall shall be calculated using Equation 9-28.

$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u \qquad \text{Equation 9-28}$$

Where:

$$P_u = P_{uw} + P_{uf} \qquad \text{Equation 9-29}$$

The deflection due to factored loads (δ_u) shall be obtained using Equation 9-30 and Equation 9-31.

(a) Where $\mathrm{M_u} < \mathrm{M_{cr}}$

$$\delta_u = \frac{5M_u h^2}{48E_m I_n} \qquad \text{Equation 9-30}$$

(b) Where $\mathrm{M_{cr}} \le \mathrm{M_u} \le \mathrm{M_n}$

$$\delta_u = \frac{5M_{cr}h^2}{48E_m I_n} + \frac{5(M_u - M_{cr})h^2}{48E_m I_{cr}} \qquad \text{Equation 9-31}$$

9.3.5.4.3 The factored moment, M_u, shall be determined either by a second-order analysis, or by a first-order analysis and Equation 9-32 through Equation 9-34.

$$M_u = \psi M_{u,0} \qquad \text{Equation 9-32}$$

Where $M_{u,o}$ is the factored moment from first-order analysis.

$$\psi = \frac{1}{1 - \frac{P_u}{P_e}} \qquad \text{Equation 9-33}$$

Where:

$$P_e = \frac{\pi^2 E_m I_{eff}}{h^2} \qquad \text{Equation 9-34}$$

For $M_u < M_{cr}$, I_{eff} shall be taken as $0.75I_n$. For $M_u \ge M_{cr}$, I_{eff} shall be taken as $I_{cr} P_u/P_e$ cannot exceed 1.0.

9.3.5.4.4 The cracking moment of the wall shall be calculated using the modulus of rupture, f_r, taken from Table 9.1.

9.3.5.4.5 The neutral axis for determining the cracked moment of inertia, I_{cr}, shall be determined in accordance with the design assumptions of Section 9.3.2. The effects of axial load shall be permitted to be included when calculating I_{cr}.

Unless stiffness values are obtained by a more comprehensive analysis, the cracked moment of inertia for a wall that is partially or fully grouted and whose neutral axis is in the face shell shall be obtained from Equation 9-35 and Equation 9-36.

$$I_{cr} = n\left(A_s + \frac{P_u}{f_y}\frac{t_{sp}}{2d}\right) \times (d-c)^2 + \frac{bc^3}{3} \qquad \text{Equation 9-35}$$

$$c = \frac{A_s f_y + P_u}{0.64 f'_m b} \qquad \text{Equation 9-36}$$

9.3.5.5 Deflections — The horizontal mid height deflection, δ_s, under allowable stress design load combinations shall be limited by the relation:

$$\delta_s < 0.007h \qquad \text{Equation 9-37}$$

P-delta effects shall be included in deflection calculation using either Section 9.3.5.5.1 or Section 9.3.5.5.2.

9.3.5.5.1 For simple support conditions top and bottom, the mid height deflection, δ_s, shall be calculated using either Equation 9-30 or Equation 9-31, as applicable, and replacing M_u with M_{ser} and δ_u with δ_s.

9.3.5.5.2 The deflection, δ_s, shall be determined by a second-order analysis that includes the effects of cracking, or by a first-order analysis with the calculated deflections magnified by a factor of $1/(1 - P/P_e)$, where P_e is determined from Equation 9-34.

9.3.6 Wall design for in-plane loads

9.3.6.1 Scope — The requirements of Section 9.3.6 shall apply to the design of walls to resist in-plane loads.

9.3.6.2 Reinforcement — Reinforcement shall be provided perpendicular to the shear reinforcement and shall be at least equal to one-third. The reinforcement shall be uniformly distributed and shall not exceed a spacing of 2.40 m.

9.3.6.3 Flexural and axial strength — The nominal flexural and axial strength shall be determined in accordance with Section 9.3.4.1.1.

9.3.6.4 Shear strength — The nominal shear strength shall be calculated in accordance with Section 9.3.4.1.2.

9.3.6.5 The maximum reinforcement requirements of Section 9.3.3.5 shall not apply if a shear wall is designed to satisfy the requirements of 9.3.6.5.1 through 9.3.6.5.5.

9.3.6.5.1 Special boundary elements need not be provided in shear walls meeting the following conditions:

(1) $P_u \le 0.10 A_g f'_m$ for geometrically symmetrical wall sections

$P_u \le 0.05 A_g f'_m$ for geometrically unsymmetrical wall sections; and either

(2) $\frac{M_u}{V_u d_v} \le 1.0$; or

(3) $V_u \le 0.25 A_{nv}\sqrt{f'_m}$ and $\frac{M_u}{V_u d_v} \le 3.0$

9.3.6.5.2 The need for special boundary elements at the edges of shear walls shall be evaluated in accordance with Section 9.3.6.5.3 or 9.3.6.5.4. The requirements of Section 9.3.6.5.5 shall also be satisfied.

9.3.6.5.3 This Section applies to walls bending in single curvature in which the flexural limit state response is governed by yielding at the base of the wall. Walls not satisfying those requirements shall be designed in accordance with Section 9.3.6.5.4.

(a) Special boundary elements shall be provided over portions of compression zones where:

$$c \ge \frac{l_w}{600(C_d \delta_{ne}/h_w)}$$

and c is calculated for the P_u given by SBC 301 Strength Design Load Combination 5 $(1.2D + 1.0E + L + 0.2S)$ or the corresponding strength design load combination of SBC301, and the corresponding nominal moment strength, M_n, at the base critical section. The load factor on L in Combination 5 is reducible to 0.5, as per exceptions to Section 2.3.2 of SBC 301.

(b) Where special boundary elements are required by Section 9.3.6.5.3(a), the special boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of l_w, or $M_u/4V_u$.

9.3.6.5.4 Shear walls not designed by Section 9.3.6.5.3 shall have special boundary elements at boundaries and edges around openings in shear walls where the maximum extreme fiber compressive stress, corresponding to factored forces including earthquake effect, exceeds $0.2f'_m$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15f'_m$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in Section 5.1.1.2.3 shall be used.

9.3.6.5.5 Where special boundary elements are required by Section 9.3.6.5.3 or 9.3.6.5.4, requirements (a) through (d) in this section shall be satisfied and tests shall be performed to verify the strain capacity of the element:

(a) The special boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of $(c - 0.1l_w)$ and $c/2$.
(b) In flanged sections, the special boundary element shall include the effective flange width in compression and shall extend at least 300 mm into the web.
(c) Special boundary element transverse reinforcement at the wall base shall extend into the support a minimum of the development length of the largest longitudinal reinforcement in the boundary element unless the special boundary element terminates on a footing or mat, where special boundary element transverse reinforcement shall extend at least 300 mm into the footing or mat.
(d) Horizontal shear reinforcement in the wall web shall be anchored to develop the specified yield strength, f_y, within the confined core of the boundary element.

9.3.6.6 Shear keys — The surface of concrete upon which a special reinforced masonry shear wall is constructed shall have a minimum surface roughness of 3 mm. Shear keys are required where the calculated tensile strain in vertical reinforcement from in-plane loads exceeds the yield strain under load combinations that include seismic forces based on an *R* factor equal to 1.5. Shear keys that satisfy the following requirements shall be placed at the interface between the wall and the foundation.

(1) The width of the keys shall be at least equal to the width of the grout space.

(2) The depth of the keys shall be at least 40 mm.

(3) The length of the key shall be at least 150 mm.

(4) The spacing between keys shall be at least equal to the length of the key.

(5) The cumulative length of all keys at each end of the shear wall shall be at least 10 percent of the length of the shear wall (20 percent total).

(6) At least 150 mm of a shear key shall be placed within 400 mm of each end of the wall.

(7) Each key and the grout space above each key in the first course of masonry shall be grouted solid.

TABLES OF CHAPTER 9

Table 9.1: Modulus of rupture, f_r, kPa

Direction of flexural tensile stress and masonry type	Mortar types			
	Portland cement/lime or mortar cement		Masonry cement or air entrained Portland cement/lime	
	M or S	N	M or S	N
Normal to bed joints				
Solid units	919	690	552	349
Hollow units[1]				
Ungrouted	579	441	349	211
Fully grouted	1124	1089	1055	1000
Parallel to bed joints in running bond				
Solid units	1839	1379	1103	689
Hollow units				
Ungrouted and partially grouted	1149	873	689	441
Fully grouted	1839	1379	1103	689
Parallel to bed joints in masonry not laid in running bond				
Continuous grout section parallel to bed	2310	2310	2310	2310
Other	0	0	0	0

1. For partially grouted masonry, modulus of rupture values shall be determined on the basis of linear interpolation between fully grouted hollow units and ungrouted hollow units based on amount (percentage) of grouting.

CHAPTER 10—PRESTRESSED MASONRY

Prestressed Masonry requirements were omitted from this version of the code as they may not be of practical use in Saudi Arabia.

CHAPTER 11—STRENGTH DESIGN OF AUTOCLAVED AERATED CONCRETE (AAC) MASONRY

11.1—General

11.1.1 Scope

This Chapter provides minimum requirements for design of AAC masonry.

11.1.1.1 Except as stated elsewhere in this Chapter, design of AAC masonry shall comply with the requirements of Part 1 and Part 2, excluding Sections 5.5.1, 5.5.2(d) and 5.3.2.

11.1.1.2 Design of AAC masonry shall comply with Sections 11.1.2 through 11.1.9, and either Section 11.2 or 11.3.

11.1.2 Required strength

Required strength shall be determined in accordance with the strength design load combinations of SBC 301. Members subject to compressive axial load shall be designed for the maximum design moment accompanying the axial load. The factored moment, M_u, shall include the moment induced by relative lateral displacement.

11.1.3 Design strength

AAC masonry members shall be proportioned so that the design strength equals or exceeds the required strength. Design strength is the nominal strength multiplied by the strength-reduction factor, ϕ, as specified in Section 11.1.5.

11.1.4 Strength of joints

AAC masonry members shall be made of AAC masonry units. The tensile bond strength of AAC masonry joints shall not be taken greater than the limits of Section 11.1.8.3. When AAC masonry units with a maximum height of 200 mm (nominal) are used, head joints shall be permitted to be left unfilled between AAC masonry units laid in running bond, provided that shear capacity is calculated using the formulas of this Code corresponding to that condition. Open head joints shall not be permitted in AAC masonry not laid in running bond.

11.1.5 Strength-reduction factors

11.1.5.1 Anchor bolts — For cases where the nominal strength of an anchor bolt is controlled by AAC masonry breakout, ϕ shall be taken as 0.50. For cases where the nominal strength of an anchor bolt is controlled by anchor bolt steel, ϕ shall be taken as 0.90. For cases where the nominal strength of an anchor bolt is controlled by anchor pullout, ϕ shall be taken as 0.65.

11.1.5.2 Bearing — For cases involving bearing on AAC masonry, ϕ shall be taken as 0.60.

11.1.5.3 Combinations of flexure and axial load in unreinforced AAC masonry — the value of ϕ shall be taken as 0.60 for unreinforced AAC masonry designed to resist flexure, axial load, or combinations thereof.

11.1.5.4 Combinations of flexure and axial load in reinforced AAC masonry — The value of ϕ shall be taken as 0.90 for reinforced AAC masonry designed to resist flexure, axial load, or combinations thereof.

11.1.5.5 Shear — The value of ϕ shall be taken as 0.80 for AAC masonry designed to resist shear.

11.1.6 Deformation requirements

11.1.6.1 Deflection of unreinforced (plain) AAC masonry — Deflection calculations for unreinforced (plain) AAC masonry members shall be based on uncracked section properties.

11.1.6.2 Deflection of reinforced AAC masonry — Deflection calculations for reinforced AAC masonry members shall be based on cracked section properties including the reinforcement and grout. The flexural and shear stiffness properties assumed for deflection calculations shall not exceed one-half of the gross section properties unless a cracked-section analysis is performed.

11.1.7 Anchor bolts

Headed and bent-bar anchor bolts shall be embedded in grout, and shall be designed in accordance with Section 9.1.6 using f'_g instead of f'_m, and neglecting the contribution of AAC to the edge distance and embedment depth. Anchors embedded in AAC without grout shall be designed using nominal capacities provided by the anchor manufacturer and verified by an independent testing agency.

11.1.8 Material properties

11.1.8.1 Compressive strength

11.1.8.1.1 Masonry compressive strength — The specified compressive strength of AAC masonry, f'_{AAC} shall equal or exceed 2.0 MPa

11.1.8.1.2 Grout compressive strength — The specified compressive strength of grout, f'_g, shall equal or exceed 14.0 MPa and shall not exceed 34 MPa.

11.1.8.2 Masonry splitting tensile strength — The splitting tensile strength fc shall be determined by Equation 11-1.

$$f_{t\,AAC} = 0.199\sqrt{f'_{AAC}} \qquad \text{Equation 11-1}$$

11.1.8.3 Masonry modulus of rupture — The modulus of rupture, f_{rAAC}, for AAC masonry elements shall be taken as twice the masonry splitting tensile strength, f_{tAAC}. If a section of AAC masonry contains a Type M or Type S horizontal leveling bed of mortar, the value of f_{rAAC} shall not exceed 345 kPa at that section. If a section of AAC masonry contains a horizontal bed joint of thin-bed mortar and AAC, the value of f_{rAAC} shall not exceed 550 kPa at that section.

11.1.8.4 Masonry direct shear strength — The direct shear strength, f_v across an interface of AAC material shall be determined by Equation 11-2, and shall be taken as 345 kPa across an interface between grout and AAC material.

$$f_v = 0.15 f'_{AAC} \qquad \text{Equation 11-2}$$

11.1.8.5 Coefficient of friction — The coefficient of friction between AAC and AAC shall be 0.75. The coefficient of friction between AAC and thin-bed mortar or between AAC and leveling-bed mortar shall be 1.0.

11.1.8.6 Reinforcement strength — Masonry design shall be based on a reinforcement strength equal to the specified yield strength of reinforcement, f_y, which shall not exceed 414 MPa. The actual yield strength shall not exceed 1.3 multiplied by the specified yield strength.

11.1.9 Nominal bearing strength

11.1.9.1 The nominal bearing strength of AAC masonry shall be calculated as f'_{AAC} multiplied by the bearing area, A_{br}, as defined in Section 4.3.4.

11.1.9.2 Bearing for simply supported precast floor and roof members on AAC masonry shear walls — The following minimum requirements shall apply so that after the consideration of tolerances, the distance from the edge of the supporting wall to the end of the precast member in the direction of the span is at least:

(a) For AAC floor panels, 50 mm
(b) For solid or hollow-core slabs, 50 mm
(c) For beams or stemmed members, 75 mm

11.1.10 Corbels — Load-bearing corbels of AAC masonry shall not be permitted. Non-load-bearing corbels of AAC masonry shall conform to the requirements of Section 5.5.2(a) through 5.5.2(c). The back section of the corbelled section shall remain within 6.5 mm of plane.

11.2—Unreinforced (plain) AAC masonry

11.2.1 Scope

The requirements of Section 11.2 are in addition to the requirements of Part 1, Part 2, and Section 11.1, and govern masonry design in which AAC masonry is used to resist tensile forces.

11.2.1.1 Strength for resisting loads — Unreinforced (plain) AAC masonry members shall be designed using the strength of masonry units, mortar, and grout in resisting design loads.

11.2.1.2 Strength contribution from reinforcement — Stresses in reinforcement shall not be considered effective in resisting design loads.

11.2.1.3 Design criteria — Unreinforced (plain) AAC masonry members shall be designed to remain uncracked.

11.2.2 Flexural strength of unreinforced (plain) AAC masonry members

The following assumptions shall apply when determining the flexural strength of unreinforced (plain) AAC masonry members:

(a) Strength design of members for factored flexure and axial load shall be in accordance with principles of engineering mechanics.
(b) Strain in masonry shall be directly proportional to the distance from the neutral axis.

(c) Flexural tension in masonry shall be assumed to be directly proportional to strain.
(d) Flexural compressive stress in combination with axial compressive stress in masonry shall be assumed to be directly proportional to strain. Nominal compressive strength shall not exceed a stress corresponding to $0.85f'_{AAC}$
(e) The nominal flexural tensile strength of AAC masonry shall be determined from Section 11.1.8.3.

11.2.3 Nominal axial strength of unreinforced (plain) AAC masonry members

Nominal axial strength, P_n, shall be calculated using Equation 11-3 or Equation 11-4.

(a) For members having an h/r ratio not greater than 99:

$$P_n = 0.8\left\{\frac{0.85A_n f'_{AAC}}{1} \times \left[1-\left(\frac{h}{140r}\right)^2\right]\right\} \quad \text{Equation 11-3}$$

(b) For members having an h/r ratio greater than 99:

$$P_n = 0.8\left\{\frac{0.85A_n f'_{AAC}}{1} \times \left[\left(\frac{70r}{h}\right)^2\right]\right\} \quad \text{Equation 11-4}$$

11.2.4 Axial tension

The tensile strength of unreinforced AAC masonry shall be neglected in design when the masonry is subjected to axial tension forces.

11.2.5 Nominal shear strength of unreinforced (plain) AAC masonry members

The nominal shear strength of AAC masonry, V_{nAAC}, shall be the least of the values calculated by Sections 11.3.4.1.2.1 through 11.3.4.1.2.3. In evaluating nominal shear strength by Section 11.3.4.1.2.3, effects of reinforcement shall be neglected. The provisions of 11.3.4.1.2 shall apply to AAC shear walls not laid in running bond. The provisions of Section 11.3.4.1.2.4 shall apply to AAC walls loaded out-of-plane.

11.2.6 Flexural cracking

The flexural cracking strength shall be calculated in accordance with Section 11.3.6.5.

11.3—Reinforced AAC masonry

11.3.1 Scope

The requirements of this section are in addition to the requirements of Part 1, Part 2, and Section 11.1 and govern AAC masonry design in which reinforcement is used to resist tensile forces.

11.3.2 Design assumptions

The following assumptions apply to the design of reinforced AAC masonry:

(a) There is strain compatibility between the reinforcement, grout, and AAC masonry.
(b) The nominal strength of reinforced AAC masonry cross sections for combined flexure and axial load shall be based on applicable conditions of equilibrium.
(c) The maximum usable strain, ε_{mu}, at the extreme AAC masonry compression fiber shall be assumed to be 0.0012 for Class 2 AAC masonry and 0.003 for Class 4 AAC masonry and higher.
(d) Strain in reinforcement and AAC masonry shall be assumed to be directly proportional to the distance from the neutral axis.
(e) Tension and compression stresses in reinforcement shall be calculated as the product of steel modulus of elasticity, E_s, and steel strain, ε_s, but shall not be greater than f_y. Except as permitted in Section 11.3.3.5 for determination of maximum area of flexural reinforcement, the compressive stress of steel reinforcement shall be neglected unless lateral restraining reinforcement is provided in compliance with the requirements of Section 5.3.1.4.
(f) The tensile strength of AAC masonry shall be neglected in calculating axial and flexural strength.
(g) The relationship between AAC masonry compressive stress and masonry strain shall be assumed to be defined by the following: AAC masonry stress of $0.85f'_{AAC}$ shall be assumed uniformly distributed over an equivalent compression stress block bounded by edges of the cross section and a straight line parallel to the neutral axis and located at a distance $a = 0.67c$ from the fiber of maximum compressive strain. The distance c from the fiber of maximum strain to the neutral axis shall be measured perpendicular to the neutral axis.

11.3.3 Reinforcement requirements and details

11.3.3.1 Reinforcing bar size limitations — Reinforcing bars used in AAC masonry shall not be larger than Dia 28. The nominal bar diameter shall not exceed one-eighth of the nominal member thickness and shall not exceed one-quarter of the least clear dimension of the grout space in which it is placed. In plastic hinge zones, the area of reinforcing bars placed in a grout space shall not exceed 3 percent of the grout space area. In other than plastic hinge zones, the area of reinforcing bars placed in a grout space shall not exceed 4.5 percent of the grout space area.

11.3.3.2 Standard hooks — The equivalent embedment length to develop standard hooks in tension, l_e, shall be determined by Equation 11-5:

$$l_e = 13d_b \qquad \text{Equation 11-5}$$

11.3.3.3 Development

11.3.3.3.1 Development of tension and compression reinforcement — the required tension or compression reinforcement shall be developed in accordance with the following provisions:

The required development length of reinforcement shall be determined by Equation 11-6, but shall not be less than 300 mm.

$$l_d = \frac{1.57 d_b^2 f_y \gamma}{K_{AAC}\sqrt{f_g'}} \qquad \text{Equation 11-6}$$

K_{AAC} shall not exceed the smallest of the following: the minimum grout cover, the clear spacing between adjacent reinforcement splices, and $9d_b$.

(a) γ= 1.0 for Dia 10 through Dia 16 bars;
(b) γ = 1.3 for Dia 18 through Dia 22 bars
(c) γ = 1.5 for Dia 25 through Dia 28 bars.

11.3.3.3.2 Development of shear reinforcement — Shear reinforcement shall extend the depth of the member less cover distances.

11.3.3.3.2.1 Except at wall intersections, the end of a horizontal reinforcing bar needed to satisfy shear strength requirements of Section 11.3.4.1.2, shall be bent around the edge vertical reinforcing bar with a 180-degree hook. The ends of single-leg or U-stirrups shall be anchored by one of the following means:

(a) A standard hook plus an effective embedment of $l_d/2$. The effective embedment of a stirrup leg shall be taken as the distance between the mid-depth of the member, d/2, and the start of the hook (point of tangency).
(b) For Dia 16 bars and smaller, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of $l_d/3$. The $l_d/3$ embedment of a stirrup leg shall be taken as the distance between mid-depth of the member, d/2, and the start of the hook (point of tangency).
(c) Between the anchored ends, each bend in the continuous portion of a transverse U-stirrup shall enclose a longitudinal bar.

11.3.3.3.2.2 At wall intersections, horizontal reinforcing bars needed to satisfy shear strength requirements of Section 11.3.4.1.2 shall be bent around the edge vertical reinforcing bar with a 90-degree standard hook and shall extend horizontally into the intersecting wall a minimum distance at least equal to the development length.

11.3.3.4 Splices — Reinforcement splices shall comply with one of the following:

(a) The minimum length of lap for bars shall be 300 mm or the development length determined by Equation 11-6, whichever is greater.
(b) A welded splice shall have the bars butted and welded to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required. Welding shall conform to AWS D1.4. Reinforcement to be welded shall conform to ASTM A706, or shall be accompanied by a submittal showing its chemical analysis and carbon equivalent as required by AWS D1.4. Existing reinforcement to be welded shall conform to ASTM A706, or shall be analyzed chemically and its carbon equivalent determined as required by AWS D1.4.
(c) Mechanical splices shall have the bars connected to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required.

11.3.3.5 Maximum reinforcement percentages — The ratio of reinforcement, ρ, shall be calculated in accordance with Section 9.3.3.5 with the following exceptions:

The maximum usable strain, ε_{mu}, at the extreme masonry compression fiber shall be in accordance with Section 11.3.2(c).

The strength of the compression zone shall be calculated as 85 percent of f'_{AAC} multiplied by 67 percent of the area of the compression zone.

11.3.3.6 Bundling of reinforcing bars — Reinforcing bars shall not be bundled.

11.3.4 Design of beams, piers, and columns

Member design forces shall be based on an analysis that considers the relative stiffness of structural members. The calculation of lateral stiffness shall include the contribution of beams, piers, and columns. The effects of cracking on member stiffness shall be considered.

11.3.4.1 Nominal strength

11.3.4.1.1 Nominal axial and flexural strength — The nominal axial strength, P_n, and the nominal flexural strength, M_n, of a cross section shall be determined in accordance with the design assumptions of Section 11.3.2 and the provisions of Section 11.3.4.1. For any value of nominal flexural strength, the corresponding calculated nominal axial strength shall be modified for the effects of slenderness in accordance with Equation 11-7 or Equation 11-8. The nominal flexural strength at any section along a member shall not be less than one-fourth of the maximum nominal flexural strength at the critical section.

The nominal axial compressive strength shall not exceed Equation 11-7 or Equation 11-8, as appropriate.

(a) For members having an h/r ratio not greater than 99:

$$P_n = 0.80[0.85f'_{AAC}(A_n - A_{st}) + f_yA_{st}]\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \text{Equation 11-7}$$

(b) For members having an h/r ratio greater than 99:

$$P_n = 0.80[0.85f'_{AAC}(A_n - A_{st}) + f_yA_{st}]\left(\frac{70r}{h}\right)^2 \quad \text{Equation 11-8}$$

11.3.4.1.2 Nominal shear strength Nominal shear strength, V_n, shall be calculated using Equation 11-9 through Equation 11-12, as appropriate.

$$V_n = V_{nAAC} + V_{ns} \quad \text{Equation 11-9}$$

where V_n shall not exceed the following:

$$V_n = \mu_{AAC}P_u \quad \text{Equation 11-10}$$

At an interface of AAC and thin-bed mortar or leveling-bed mortar, the nominal sliding shear strength shall be calculated using Equation 11-10 and using the coefficient of friction from Section 11.1.8.5.

(a) Where $M_u/(V_u d_v) \le 0.25$:

$$V_n \le 0.498A_{nv}\sqrt{f'_{AAC}} \quad \text{Equation 11-11}$$

(b) Where $M_u/(V_u d_v) \ge 1.0$:

$$V_n \le 0.332A_{nv}\sqrt{f'_{AAC}} \quad \text{Equation 11-12}$$

(c) The maximum value of V_n for $M_u/(V_u\, d_v)$ between 0.25 and 1.0 shall be permitted to be linearly interpolated.

The nominal masonry shear strength shall be taken as the least of the values calculated using Section 11.3.4.1.2.1 and 11.3.4.1.2.2.

11.3.4.1.2.1 Nominal masonry shear strength as governed by web-shear cracking — Nominal masonry shear strength as governed by web-shear cracking, V_{nAAC}, shall be calculated using Equation 11-13a for AAC masonry with mortared head joints, and Equation 11-13b for masonry with unmortared head joints:

$$V_{nAAC} = 0.0789l_wt\sqrt{f'_{AAC}}\sqrt{1+\frac{P_u}{0.199\sqrt{f'_{AAC}}l_wt}} \quad \text{Equation 11-13a}$$

$$V_{nAAC} = 0.0548l_wt\sqrt{f'_{AAC}}\sqrt{1+\frac{P_u}{0.199\sqrt{f'_{AAC}}l_wt}} \quad \text{Equation 11-14b}$$

For AAC masonry not laid in running bond, nominal masonry shear strength as governed by web-shear cracking, V_{nAAC}, shall be calculated using Equation 11-13c:

$$V_{nAAC} = 0.0747\sqrt{f'_{AAC}}A_{nv} + 0.05P_u \quad \text{Equation 11 13c}$$

11.3.4.1.2.2 Nominal shear strength as governed by crushing of diagonal compressive strut — For walls with $M_u/(V_ud_v) < 1.5$, nominal shear strength, V_{nAAC}, as governed by crushing of a diagonal strut, shall be calculated as follows:

$$V_{nAAC} = 0.17 f'_{AAC} t \frac{h.l_w^2}{h^2 + \left(\frac{3}{4} l_w\right)^2} \qquad \text{Equation 11-15}$$

For walls with $M_u / (V_u d_v)$ equal to or exceeding 1.5, capacity as governed by crushing of the diagonal compressive strut need not be calculated.

11.3.4.1.2.3 Nominal shear strength provided by shear reinforcement — Nominal shear strength provided by reinforcement, V_{ns}, shall be calculated as follows:

$$V_{ns} = 0.50 \left(\frac{A_v}{s}\right) f_y d_v \qquad \text{Equation 11-16}$$

Nominal shear strength provided by reinforcement, V_{ns}, shall include only deformed reinforcement embedded in grout for AAC shear walls.

11.3.4.1.2.4 Nominal shear strength for beams and for out-of-plane loading of other members shall be calculated as follows:

$$V_{nAAC} = 0.066 \sqrt{f'_{AAC}} bd \qquad \text{Equation 11-17}$$

11.3.4.2 Beams — Design of beams shall meet the requirements of Section 5.2 and the additional requirements of Sections 11.3.4.2.1 through 11.3.4.2.5.

11.3.4.2.1 The factored axial compressive force on a beam shall not exceed $0.05 A_n f'_{AAC}$.

11.3.4.2.2 Longitudinal reinforcement

11.3.4.2.2.1 The variation in longitudinal reinforcing bars shall not be greater than one bar size. Not more than two bar sizes shall be used in a beam.

11.3.4.2.2.2 The nominal flexural strength of a beam shall not be less than 1.3 multiplied by the nominal cracking moment of the beam, M_{cr}. The modulus of rupture, f_{rAAC}, for this calculation shall be determined in accordance with Section 11.1.8.3.

11.3.4.2.3 Transverse reinforcement — Transverse reinforcement shall be provided where V_u exceeds ϕV_{nAAC}. The factored shear, V_u, shall include the effects of lateral load. When transverse reinforcement is required, the following provisions shall apply:

(a) Transverse reinforcement shall be a single bar with a 180-degree hook at each end.
(b) Transverse reinforcement shall be hooked around the longitudinal reinforcement.
(c) The minimum area of transverse reinforcement shall be $0.0007bd_v$.
(d) The first transverse bar shall not be located more than one-fourth of the beam depth, d_v, from the end of the beam.
(e) The maximum spacing shall not exceed the lesser of one-half the depth of the beam or 1200 mm.

11.3.4.2.4 Construction — Beams shall be fully grouted.

11.3.4.2.5 Dimensional limits — The nominal depth of a beam shall not be less than 200 mm.

11.3.4.3 Piers

11.3.4.3.1 The factored axial compression force on the piers shall not exceed $0.3 A_n f'_{AAC}$.

11.3.4.3.2 Longitudinal reinforcement — A pier subjected to in-plane stress reversals shall be reinforced symmetrically about the geometric center of the pier. The longitudinal reinforcement of piers shall comply with the following:

(a) At least one bar shall be provided in each end cell.
(b) The minimum area of longitudinal reinforcement shall be 0.0007bd.

11.3.4.3.3 Dimensional limits — Dimensions shall be in accordance with the following:

(a) The nominal thickness of a pier shall not be less than 150 mm and shall not exceed 400 mm.
(b) The distance between lateral supports of a pier shall not exceed 25 multiplied by the nominal thickness of a pier except as provided for in Section 11.3.4.3.3(c).
(c) When the distance between lateral supports of a pier exceeds 25 multiplied by the nominal thickness of the pier, design shall be based on the provisions of Section 11.3.5.
(d) The nominal length of a pier shall not be less than three multiplied by its nominal thickness nor greater than six multiplied by its nominal thickness. The clear height of a pier shall not exceed five multiplied by its nominal length.
(e) Exception: When the factored axial force at the location of maximum moment is less than $0.05 f'_{AAC} A_g$, the length of a pier shall

be permitted to be taken equal to the thickness of the pier.

11.3.5 Wall design for out-of-plane loads

11.3.5.1 Scope — The requirements of Section 11.3.5 shall apply to the design of walls for out-of-plane loads.

11.3.5.2 Maximum reinforcement — The maximum reinforcement ratio shall be determined by Section 11.3.3.5.

11.3.5.3 Nominal axial and flexural strength the nominal axial strength, P_n, and the nominal flexural strength, M_u, of a cross-section shall be determined in accordance with the design assumptions of Section 11.3.2. The nominal axial compressive strength shall not exceed that determined by Equation 11-7 or Equation 11-8, as appropriate.

11.3.5.4 Nominal shear strength — The nominal shear strength shall be determined by Section 11.3.4.1.2.

11.3.5.5 P-Delta effects

11.3.5.5.1 Members shall be designed for the factored axial load, P_u, and the moment magnified for the effects of member curvature, M_u. The magnified moment shall be determined either by Section 11.3.5.5.2 or Section 11.3.5.5.3.

11.3.5.5.2 Moment and deflection calculations in this Section are based on simple support conditions top and bottom. For other support and fixity conditions, moments, and deflections shall be calculated using established principles of mechanics.

The procedures set forth in this section shall be used when the factored axial load stress at the location of maximum moment satisfies the requirement calculated by Equation 11-18.

$$\frac{P_u}{A_g} \le 0.2 f'_{AAC} \qquad \text{Equation 11-18}$$

When the ratio of effective height to nominal thickness, h/t, exceeds 30, the factored axial stress shall not exceed $0.05 f'_{AAC}$.

Factored moment and axial force shall be determined at the midheight of the wall and shall be used for design. The factored moment, M_u, at the midheight of the wall shall be calculated using Equation 11-19.

$$M_u = \frac{w_u h^2}{8} + P_u \delta_u + P_{uf}\frac{e_u}{2} \qquad \text{Equation 11-19}$$

Where:

$$P_u = P_{uw} + P_{uf} \qquad \text{Equation 11-20}$$

The deflection due to factored loads (δ_u) shall be obtained using Equation 11-21 and Equation 11-22.

(a) Where $M_u < M_{cr}$

$$\delta_u = \frac{5 M_u h^2}{48 E_{AAC} I_n} \qquad \text{Equation 11-21}$$

(b) Where $M_{cr} \le M_u \le M_n$

$$\delta_u = \frac{5 M_{cr} h^2}{48 E_{AAC} I_n} + \frac{5(M_u - M_{cr})h^2}{48 E_{AAC} I_{cr}} \qquad \text{Equation 11-22}$$

11.3.5.5.3 The factored moment, M_u, shall be determined either by a second-order analysis, or by a first-order analysis and Equation 11-23 through Equation 11-25.

$$M_u = \psi M_{u,0} \qquad \text{Equation 11-23}$$

Where $M_{u,0}$ is the factored moment from first-order analysis.

$$\psi = \frac{1}{1 - \frac{P_u}{P_e}} \qquad \text{Equation 11-24}$$

Where:

$$P_e = \frac{\pi^2 E_{AAC} I_{eff}}{h^2} \qquad \text{Equation 11-25}$$

For $M_u < M_{cr}$, I_{eff} shall be taken as $0.75 I_n$. For $M_u > M_{cr}$, I_{eff} shall be taken as I_{cr}. P_u/P_e cannot exceed 1.0.

11.3.5.5.4 The cracking moment of the wall shall be calculated using Equation 11-26, where f_{rAAC} is given by Section 11.1.8.3:

$$M_{cr} = S_n\left(f_{rAAC} + \frac{P}{A_n}\right) \qquad \text{Equation 11-26}$$

If the section of AAC masonry contains a horizontal leveling bed, the value of f_{rAAC} shall not exceed 345 kPa.

11.3.5.5.5 The neutral axis for determining the cracked moment of inertia, I_{cr}, shall be determined in accordance with the design assumptions of Section 11.3.2. The effects of axial load shall be permitted to be included when calculating I_{cr}.

Unless stiffness values are obtained by a more comprehensive analysis, the cracked moment of inertia for a solidly grouted wall or a partially grouted wall with the neutral axis in the face shell shall be obtained from Equation 11-27 and Equation 11-28.

$$I_{cr} = n\left(A_s + \frac{P_u}{f_y}\frac{t_{sp}}{2d}\right)(d - c)^2 + \frac{b(c)^3}{3} \quad \text{Equation 11-27}$$

$$c = \frac{A_s f_y + P_u}{0.57 f'_{AAC} b} \quad \text{Equation 11-28}$$

11.3.5.5.6 The design strength for out-of-plane wall loading shall be in accordance with Equation 11-29.

$$M_u \le \phi M_n \quad \text{Equation 11-29}$$

The nominal moment shall be calculated using Equation 11-30 and Equation 11-31 if the reinforcing steel is placed in the center of the wall.

$$M_n = (A_s f_y + P_u)\left(d - \frac{a}{2}\right) \quad \text{Equation 11-30}$$

$$a = \frac{(P_u + A_s f_y)}{0.85 f'_{AAC} b} \quad \text{Equation 11-31}$$

11.3.5.6 Deflections — The horizontal midheight deflection, δ_s, under allowable stress design load combinations shall be limited by the relation:

$$\delta_s \le 0.007h \quad \text{Equation 11-32}$$

P-delta effects shall be included in deflection calculation using either Section 11.3.5.6.1 or Section 11.3.5.6.2.

11.3.5.6.1 For simple support condition top and bottom, the midheight deflection, δ_s, shall be calculated using either Equation 11-21 or Equation 11-22, as applicable, and replacing M_u with M_{ser}, and δ_u with δ_s.

11.3.5.6.2 The deflection, δ_s, shall be determined by a second-order analysis that includes the effects of cracking, or by a first-order analysis with the calculated deflections magnified by a factor of $1/(1 - P/P_e)$, where P_e is determined from Equation 11-25.

11.3.6 Wall design for in-plane loads

11.3.6.1 Scope — the requirements of Section 11.3.6 shall apply to the design of walls to resist in-plane loads.

11.3.6.2 Reinforcement — Reinforcement shall be in accordance with the following:

(a) Reinforcement shall be provided perpendicular to the shear reinforcement and shall be at least equal to one-third A_v. The reinforcement shall be uniformly distributed and shall not exceed a spacing of 2.45 m.

(b) The maximum reinforcement ratio shall be determined in accordance with Section 11.3.3.5.

11.3.6.3 Flexural and axial strength — The nominal flexural and axial strength shall be determined in accordance with Section 11.3.4.1.1.

11.3.6.4 Shear strength — The nominal shear strength shall be calculated in accordance with Section 11.3.4.1.2.

11.3.6.5 Flexural cracking strength — The flexural cracking strength shall be calculated in accordance with Equation 11-33, where f_{rAAC} is given by Section 11.1.8.3:

$$V_{cr} = \frac{S_n}{h}\left(f_{rAAC} + \frac{P}{A_n}\right) \quad \text{Equation 11-33}$$

If the section of AAC masonry contains a horizontal leveling bed, the value of f_{rAAC} shall not exceed 345 kPa.

11.3.6.6 The maximum reinforcement requirements of Section 11.3.3.5 shall not apply if a shear wall is designed to satisfy the requirements of Sections 11.3.6.6.1 through 11.3.6.6.4.

11.3.6.6.1 The need for special boundary elements at the edges of shear walls shall be evaluated in accordance with Section 11.3.6.6.2 or 11.3.6.6.3. The requirements of Section 11.3.6.6.4 shall also be satisfied.

11.3.6.6.2 This Section applies to walls bending in single curvature in which the flexural limit state response is governed by yielding at the base of the wall. Walls not satisfying those requirements shall be designed in accordance with Section 11.3.6.6.3.

(a) Special boundary elements shall be provided over portions of compression zones where:

$$c \geq \frac{l_w}{600(C_d \delta_{ne}/h_w)} \quad \text{Equation 11-34}$$

and c is calculated for the P_u given by SBC 301 Load Combination 5 $(1.2D + 1.0E + L + 0.2S)$, and the corresponding nominal moment strength, M_n, at the base critical section. The load factor on L in Load Combination 5 is reducible to 0.5, as per exceptions to Section 2.3.2 of SBC 301.

(b) Where special boundary elements are required by Section 11.3.6.6.2 (a), the special boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of l_w, or $M_u/4V_u$.

11.3.6.6.3 Shear walls not designed to the provisions of Section 11.3.6.6.2 shall have special 9.3.6.5.4. boundary elements at boundaries and edges around openings in shear walls where the maximum extreme fiber compressive stress, corresponding to factored forces including earthquake effect, exceeds $0.2f'_{AAC}$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15f'_{AAC}$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in Section 5.1.1.2.3 shall be used.

11.3.6.6.4 Where special boundary elements are required by Section 11.3.6.6.2 or 11.3.6.6.3, 9.3.6.5.5(a) through (d) shall be satisfied and tests shall be performed to verify the strain capacity of the element:

(a) The special boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of $(c - 0.1l_w)$ and $c/2$.

(b) In flanged sections, the special boundary element shall include the effective flange width in compression and shall extend at least 300 mm into the web.

(c) Special boundary element transverse reinforcement at the wall base shall extend into the support at least the development length of the largest longitudinal reinforcement in the boundary element unless the special boundary element terminates on a footing or mat, where special boundary element transverse reinforcement shall extend at least 300 mm into the footing or mat.

(d) Horizontal shear reinforcement in the wall web shall be anchored to develop the specified yield strength, f_y, within the confined core of the boundary element.

PART 4—PRESCRIPTIVE DESIGN METHOD

CHAPTER 12—VENEER

12.1—General

12.1.1 Scope

This chapter provides requirements for design and detailing of anchored masonry veneer and adhered masonry veneer.

12.1.1.1 The provisions of Part 1, excluding Sections 1.2.1(c) and 1.2.2; Chapter 4, excluding Sections 4.1 and 4.3, and Chapter 6 shall apply to design of anchored and adhered veneer except as specifically stated in this Chapter.

12.1.1.2 Section 4.5 shall not apply to adhered veneer.

12.1.1.3 Articles 1.4 A and B and 3.4 C of TMS 602/ACI 530.1/ASCE 6 shall not apply to any veneer. Articles 3.4 B and F shall not apply to anchored veneer. Articles 3.3 B and 3.4 A, B, E and F shall not apply to adhered veneer.

12.1.2 Design of anchored veneer

Anchored veneer shall meet the requirements of Section 12.1.6 and shall be designed rationally by Section 12.2.1 or detailed by the prescriptive requirements of Section 12.2.2.

12.1.3 Design of adhered veneer

Adhered veneer shall meet the requirements of Section 12.1.6, and shall be designed rationally by Section 12.3.1 or detailed by the prescriptive requirements of Section 12.3.2.

12.1.4 Dimension stone

The provisions of Sections 12.1.1, 12.1.3 and 12.3 shall apply to design of adhered dimension stone veneer. Anchored dimension stone veneer is not addressed by this Code. Such a veneer system shall be considered a Special System, and consideration for approval of its use shall be submitted to the Building Official.

12.1.5 Autoclaved aerated concrete masonry veneer

Autoclaved aerated concrete masonry as a veneer wythe is not addressed by this Chapter. Such a veneer system shall be considered a Special System, and consideration for approval of its use shall be submitted to the Building Official.

12.1.6 General design requirements

12.1.6.1 Design and detail the backing system of exterior veneer to resist water penetration. Exterior sheathing shall be covered with a water-resistant membrane, unless the sheathing is water resistant and the joints are sealed.

12.1.6.2 Design and detail flashing and weep holes in exterior veneer wall systems to resist water penetration into the building interior. Weep holes shall be at least 5 mm in diameter and spaced less than 800 mm on center.

12.1.6.3 Design and detail the veneer to accommodate differential movement.

12.2—Anchored veneer

12.2.1 Alternative design of anchored masonry veneer

The alternative design of anchored veneer, which is permitted under Section 1.3, shall satisfy the following conditions:

(a) Loads shall be distributed through the veneer to the anchors and the backing using principles of mechanics.
(b) Out-of-plane deflection of the backing shall be limited to maintain veneer stability.
(c) The veneer is not subject to the flexural tensile stress provisions of Section 8.2 or the nominal flexural tensile strength provisions of Section 9.1.9.2.
(d) The provisions of Section 12.1, Section 12.2.2.9, and Section 12.2.2.10 shall apply.

12.2.2 Prescriptive requirements for anchored masonry veneer

12.2.2.1 Except as provided in Section 12.2.2.11, prescriptive requirements for anchored masonry veneer shall not be used in areas where the velocity pressure, q_z, exceeds 1.92 kPa as given in SBC 301.

12.2.2.2 Connect anchored veneer to the backing with anchors that comply with Section 12.2.2.5 and Article 2.4 of TMS 602/ACI 530.1/ASCE 6.

12.2.2.3 Vertical support of anchored masonry veneer

12.2.2.3.1 The weight of anchored veneer shall be supported vertically on concrete or masonry foundations or other noncombustible structural construction, except as permitted in Section 12.2.2.3.1.1.

12.2.2.3.1.1 If anchored veneer with a backing of cold-formed steel framing exceeds 9.15 m, or 11.60 m at a gable, in height above the location where the veneer is supported, the weight of the veneer shall be supported by noncombustible construction at each story above 9.15 m in height

12.2.2.3.1.2 When anchored veneer is used as an interior finish on wood framing, it shall have a weight of 195 kg/m^2 or less and be installed in conformance with the provisions of this Chapter.

12.2.2.3.2 When anchored veneer is supported by floor construction, the floor shall be designed to limit deflection as required in Section 5.2.1.4.1.

12.2.2.3.3 Provide noncombustible lintels or supports attached to noncombustible framing over openings where the anchored veneer is not self-supporting. Lintels shall have a length of bearing not less than 100 mm. The deflection of such lintels or supports shall conform to the requirements of Section 5.2.1.4.1.

12.2.2.4 Masonry units — Masonry units shall be at least 67 mm in actual thickness.

12.2.2.5 Anchor requirements

12.2.2.5.1 Corrugated sheet-metal anchors

12.2.2.5.1.1 Corrugated sheet-metal anchors shall be at least 22 mm wide, have a base metal thickness of at least 0.8 mm, and shall have corrugations with a wavelength of 7.6 to 12.70 mm and an amplitude of 1.5 to 2.5 mm.

12.2.2.5.1.2 Corrugated sheet-metal anchors shall be placed as follows:

(a) With solid units, embed anchors in the mortar joint and extend into the veneer a minimum of 38.0 mm, with at least 16-mm mortar cover to the outside face.
(b) With hollow units, embed anchors in mortar or grout and extend into the veneer a minimum of 38.0 mm, with at least 16-mm mortar or grout cover to the outside face.

12.2.2.5.2 Sheet-metal anchors

12.2.2.5.2.1 Sheet-metal anchors shall be at least 22.0 mm wide, shall have a base metal thickness of at least 1.5 mm, and shall:

(a) have corrugations as given in Section 12.2.2.5.1.1, or
(b) be bent, notched, or punched to provide equivalent performance in pull-out or push-through.

12.2.2.5.2.2 Sheet-metal anchors shall be placed as follows:

(a) With solid units, embed anchors in the mortar joint and extend into the veneer a minimum of 38.0 mm, with at least 16.0 mm mortar cover to the outside face.
(b) With hollow units, embed anchors in mortar or grout and extend into the veneer a minimum of 38.0 mm, with at least 16.0 mm mortar or grout cover to the outside face.

12.2.2.5.3 Wire anchors

12.2.2.5.3.1 Wire anchors shall be at least wire size WD 4.0 and have ends bent to form an extension from the bend at least 50.0 mm long. Wire anchors shall be without drips.

12.2.2.5.3.2 Wire anchors shall be placed as follows:

(a) With solid units, embed anchors in the mortar joint and extend into the veneer a minimum of 38.0 mm, with at least 16.0 mm mortar cover to the outside face.
(b) With hollow units, embed anchors in mortar or grout and extend into the veneer a minimum of 38.0 mm, with at least 16.0 mm mortar or grout cover to the outside face.

12.2.2.5.4 Joint reinforcement

12.2.2.5.4.1 Ladder-type or tab-type joint reinforcement is permitted. Cross wires used to anchor masonry veneer shall be at least wire size WD 4.0 and shall be spaced at a maximum of 400 mm on center. Cross wires shall be welded to longitudinal wires, which shall be at least wire size WD 4.0. Cross wires and taps shall be without drips

12.2.2.5.4.2 Embed longitudinal wires of joint reinforcement in the mortar joint with at least 16.0 mm mortar cover on each side.

12.2.2.5.5 Adjustable anchors

12.2.2.5.5.1 Sheet-metal and wire components of adjustable anchors shall conform to the requirements of Section 12.2.2.5.2 or 12.2.2.5.3. Adjustable anchors with joint reinforcement shall also meet the requirements of Section 12.2.2.5.4.

12.2.2.5.5.2 Maximum clearance between connecting parts of the tie shall be 1.6 mm.

12.2.2.5.5.3 Adjustable anchors shall be detailed to prevent disengagement.

12.2.2.5.5.4 Pintle anchors shall have one or more pintle legs of wire size WD 5.0 and shall have an offset not exceeding 32.0 mm.

12.2.2.5.5.5 Adjustable anchors of equivalent strength and stiffness to those specified in Sections 12.2.2.5.5.1 through 12.2.2.5.5.4 are permitted.

12.2.2.5.6 Anchor spacing

12.2.2.5.6.1 For adjustable two-piece anchors, anchors of wire size WD 4.0, and 0.8 mm corrugated sheet-metal anchors, provide at least one anchor for each 0.25 m^2 of wall area.

12.2.2.5.6.2 For other anchors, provide at least one anchor for each 0.33 m^2 of wall area.

12.2.2.5.6.3 Space anchors at a maximum of 800 mm horizontally and 625 mm vertically, but not to exceed the applicable requirements of Section 12.2.2.5.6.1 or 12.2.2.5.6.2.

12.2.2.5.6.4 Provide additional anchors around openings larger than 400 mm in either dimension. Space anchors around perimeter of opening at a maximum of 0.90 m on center. Place anchors within 300 mm of openings.

12.2.2.5.7 Joint thickness for anchors — Mortar bed joint thickness shall be at least twice the thickness of the embedded anchor.

12.2.2.6 Masonry veneer anchored to wood backing is not a common construction practice in the Kingdom of Saudi Arabia nor in other GCC countries.

12.2.2.7 Masonry veneer anchored to steel backing

12.2.2.7.1 Attach veneer with adjustable anchors.

12.2.2.7.2 Attach each anchor to steel framing with at least a No.10 corrosion-resistant screw (nominal shank diameter of 5 mm, or with a fastener having equivalent or greater pullout strength.

12.2.2.7.3 Cold-formed steel framing shall be corrosion resistant and have a minimum base metal thickness of 1.0 mm.

12.2.2.7.4 A 115-mm maximum distance between the inside face of the veneer and the steel framing shall be specified. A 25.0 mm minimum air space shall be specified.

12.2.2.8 Masonry veneer anchored to masonry or concrete backing

12.2.2.8.1 Attach veneer to masonry backing with wire anchors, adjustable anchors, or joint reinforcement. Attach veneer to concrete backing with adjustable anchors.

12.2.2.8.2 A 115 mm maximum distance between the inside face of the veneer and the outside face of the masonry or concrete backing shall be specified. A 25.0 mm minimum air space shall be specified.

12.2.2.9 Veneer not laid in running bond — Anchored veneer not laid in running bond shall have joint reinforcement of at least one wire, of size WD 4.0, spaced at a maximum of 450 mm on center vertically.

12.2.2.10 Requirements in seismic areas

12.2.2.10.1 Seismic Design Category C

12.2.2.10.1.1 The requirements of this section apply to anchored veneer for buildings in Seismic Design Category C.

12.2.2.10.1.2 Isolate the sides and top of anchored veneer from the structure so that vertical and lateral seismic forces resisted by the structure are not imparted to the veneer.

12.2.2.10.2 Seismic Design Category D

12.2.2.10.2.1 The requirements for Seismic Design Category C and the requirements of this section apply to anchored veneer for buildings in Seismic Design Category D.

12.2.2.10.2.2 Reduce the maximum wall area supported by each anchor to 75 percent of that required in Sections 12.2.2.5.6.1 and 12.2.2.5.6.2. Maximum horizontal and vertical spacings are unchanged.

12.2.2.11 Requirements in areas of high winds — The following requirements apply in areas where the velocity pressure, q_z exceeds 1.92 kPa but does not exceed 2.63 kPa and the building's mean roof height is less than or equal to 18.3 m:

(a) Reduce the maximum wall area supported by each anchor to 70 percent of that required in Sections 12.2.2.5.6.1 and 12.2.2.5.6.2.
(b) Space anchors at a maximum 450 mm horizontally and vertically.
(c) Provide additional anchors around openings larger than 400 mm in either direction. Space anchors around perimeter of opening at a maximum of 600 mm on

center. Place anchors within 300 mm of openings.

12.2.2.11.1 Provide continuous single wire joint reinforcement of wire size MW11 at a maximum spacing of 450 mm on center vertically. Mechanically attach anchors to the joint reinforcement with clips or hooks. Corrugated sheet metal anchors shall not be used.

12.3—Adhered veneer

12.3.1 Alternative design of adhered masonry veneer

The alternative design of adhered veneer, which is permitted under Section 1.3, shall satisfy the following conditions:

(a) Loads shall be distributed through the veneer to the backing using principles of mechanics.
(b) Out-of-plane curvature shall be limited to prevent veneer unit separation from the backing.
(c) The veneer is not subject to the flexural tensile stress provisions of Section 8.2 or the nominal flexural tensile strength provisions of Section 9.1.9.2.
(d) The provisions of Section 12.1 shall apply.

12.3.2 Prescriptive requirements for adhered masonry veneer

12.3.2.1 Unit sizes — Adhered veneer units shall not exceed 66.0 mm in specified thickness, 900 mm in any face dimension, nor more than 0.46 m^2 in total face area, and shall not weigh more than 73 kg/m^2.

12.3.2.2 Wall area limitations — The height, length, and area of adhered veneer shall not be limited except as required to control restrained differential movement stresses between veneer and backing.

12.3.2.3 Backing — Backing shall provide a continuous, moisture-resistant surface to receive the adhered veneer. Backing is permitted to be masonry, concrete, or metal lath and Portland cement plaster applied to masonry, concrete, steel framing, or wood framing.

12.3.2.4 Adhesion developed between adhered veneer units and backing shall have a shear strength of at least 345 kPa based on gross unit surface area when tested in accordance with ASTM C482, or shall be adhered in compliance with Article 3.3 C of TMS 602/ ACI 530.1/ASCE 6.

CHAPTER 13—GLASS UNIT MASONRY

13.1—General

13.1.1 Scope

This chapter provides requirements for empirical design of glass unit masonry as non-load-bearing elements in exterior or interior walls.

13.1.1.1 The provisions of Part 1 and Part 2, excluding Sections 1.2.1(c), 1.2.2, 4.1, 4.2, and 4.3, shall apply to design of glass unit masonry, except as stated in this Chapter.

13.1.1.2 Article 1.4 of TMS 602/ACI 530.1/ASCE 6 shall not apply to glass unit masonry.

13.1.2 General design requirements

Design and detail glass unit masonry to accommodate differential movement.

13.1.3 Units

13.1.3.1 Hollow or solid glass block units shall be standard or thin units.

13.1.3.2 The specified thickness of standard units shall be at least 98 mm.

13.1.3.3 The specified thickness of thin units shall be 80 mm for hollow units or 75 mm for solid units.

13.2—Panel size

13.2.1 Exterior standard-unit panels

The maximum area of each individual standard-unit panel shall be based on the design wind pressure, in accordance with Figure 13.1. The maximum dimension between structural supports shall be 7.6 m horizontally or 6.10 m vertically.

13.2.2 Exterior thin-unit panels

The maximum area of each individual thin-unit panel shall be 9.29 m^2. The maximum dimension between structural supports shall be 4.50 m wide or 3.0 m high. Thin units shall not be used in applications where the factored design wind pressure per SBC 301 exceeds 1,500 Pa.

13.2.3 Interior panels

13.2.3.1 When the factored wind pressure does not exceed 768 Pa, the maximum area of each individual standard-unit panel shall be 23.22 m^2 and the maximum area of each thin-unit panel shall be 13.94 m^2. The maximum dimension between structural supports shall be 7.60 m wide or 6.10 m high.

13.2.3.2 When the factored wind pressure exceeds 768 Pa, standard-unit panels shall be designed in accordance with Section 13.2.1 and thin-unit panels shall be designed in accordance with Section 13.2.2.

13.2.4 Curved panels

The width of curved panels shall conform to the requirements of Sections 13.2.1, 13.2.2, and 13.2.3, except additional structural supports shall be provided at locations where a curved section joins a straight section and at inflection points in multi-curved walls.

13.3—Support

13.3.1 General requirements

Glass unit masonry panels shall be isolated so that in-plane loads are not imparted to the panel.

13.3.2 Vertical

13.3.2.1 Maximum total deflection of structural members supporting glass unit masonry shall not exceed $l/600$.

13.3.3 Lateral

13.3.3.1 Glass unit masonry panels, more than one unit wide or one unit high, shall be laterally supported along the top and sides of the panel. Lateral support shall be provided by panel anchors along the top and sides spaced not more than 400 mm on center or by channel-type restraints. Glass unit masonry panels shall be recessed at least 25.0 mm within channels and chases. Channel-type restraints must be oversized to accommodate expansion material in the opening, and packing and sealant between the framing restraints and the glass unit masonry perimeter units. Lateral supports for glass unit masonry panels shall be designed to resist applied loads, or a minimum of 3000 N/m of panel, whichever is greater.

13.3.3.2 Glass unit masonry panels that are no more than one unit wide shall conform to the requirements of Section 13.3.3.1, except that lateral support at the top of the panel is not required.

13.3.3.3 Glass unit masonry panels that are no more than one unit high shall conform to the requirements of Section 13.3.3.1, except that lateral support at the sides of the panels is not required.

13.3.3.4 Glass unit masonry panels that are a single glass masonry unit shall conform to the requirements of Section 13.3.3.1, except that lateral support shall not be provided by panel anchors.

13.4—Expansion joints

Glass unit masonry panels shall be provided with expansion joints along the top and sides at structural supports. Expansion joints shall have sufficient thickness to accommodate displacements of the supporting structure, but shall not be less than 10 mm in thickness. Expansion joints shall be entirely free of mortar or other debris and shall be filled with resilient material.

13.5—Base surface treatment

The surface on which glass unit masonry panels are placed shall be coated with a water-based asphaltic emulsion or other elastic waterproofing material prior to laying the first course.

13.6—Mortar

Glass unit masonry shall be laid with Type S or N mortar.

13.7—Reinforcement

Glass unit masonry panels shall have horizontal joint reinforcement spaced not more than 400 mm on center, located in the mortar bed joint, and extending the entire length of the panel but not across expansion joints. Longitudinal wires shall be lapped a minimum of 150 mm at splices. Joint reinforcement shall be placed in the bed joint immediately below and above openings in the panel. The reinforcement shall have at least two parallel longitudinal wires of size WD 4.0 and have welded cross wires of size WD 4.0.

TABLES AND FIGURES OF CHAPTER 13

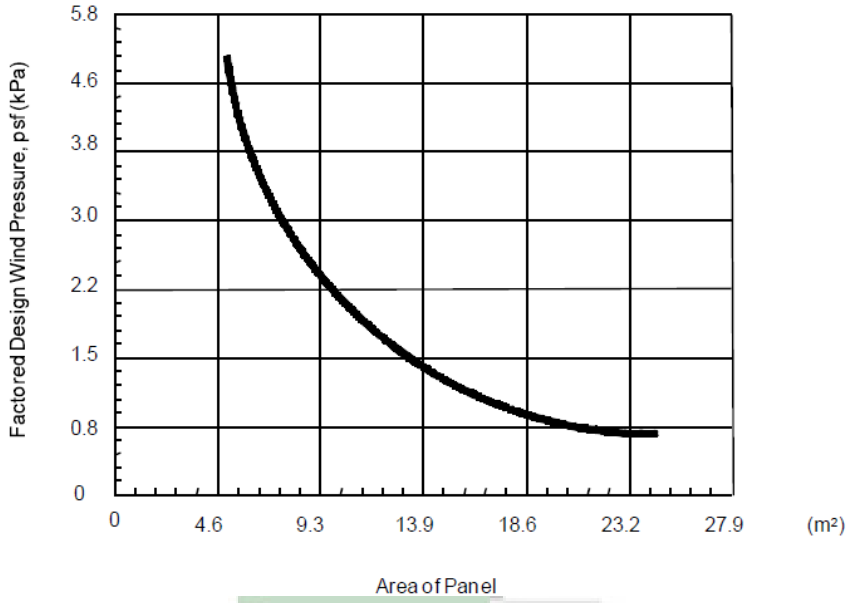


Figure 13.1: Factored design wind pressure for glass unit masonry

CHAPTER 14—MASONRY PARTITION WALLS

14.1—General

14.1.1 Scope

This chapter provides requirements for the design of masonry partition walls.

14.1.2 Design of partition walls

Partition walls shall be designed by one of the following:

(a) The prescriptive design requirements of Section 14.2 through 14.5, or
(b) The requirements of Part 1, Part 2 and the requirements of Chapter 8, Chapter 9, Chapter 11, or Chapter 13.

14.2—Prescriptive design of partition walls

14.2.1 General

14.2.1.1 The provisions of Part 1 and Part 2, excluding Sections 1.2.1(c), 1.2.2, 4.1, 4.2, and 4.3, shall apply to prescriptive design of masonry partition walls.

14.2.1.2 Article 1.4 of TMS 602/AC1 530.1/ ASCE 6 shall not apply to prescriptively designed masonry partition walls.

14.2.2 Thickness Limitations

14.2.2.1 Minimum thickness — The minimum nominal thickness of partition walls shall be 100 mm.

14.2.2.2 Maximum thickness — The maximum nominal thickness of partition walls shall be 300 mm.

14.2.3 Limitations

14.2.3.1 Vertical loads — The prescriptive design requirements of Chapter 14 shall not apply to the design of partition walls that support vertical compressive, service loads of more than 2900 N/m in addition to their own weight. The resultant of vertical loads shall be placed within the center third of the wall thickness. The prescriptive design requirements of Chapter 14 shall not apply to the design of partition walls that resist net axial tension.

14.2.3.2 Lateral loads — The prescriptive design requirements of Chapter 14 shall not apply to partition walls resisting service level unfactored lateral loads that exceed 0.240 kPa when using Table 14.1 or 0.48 kPa when using

14.2.3.3 Table 14.2.

14.2.3.4 Seismic Design Category — the prescriptive design requirements of Chapter 14 shall not apply to the design of masonry partition walls in Seismic Design Category D.

14.2.3.5 Nonparticipating Elements — Partition walls designed using the prescriptive requirements of Chapter 14 shall be designed as 'nonparticipating elements' in accordance with the requirements of Section 7.3.1.

14.2.3.6 Enclosed Buildings — The prescriptive design requirements of Chapter 14 shall only be permitted to be applied to the design of masonry partition walls in Enclosed Buildings as defined by SBC 301

14.2.3.7 Risk Category IV — The prescriptive design requirements of Chapter 14 shall not apply to the design of masonry partition walls in Risk Category IV as defined in SBC 301.

14.2.3.8 Masonry not laid in running bond — The prescriptive design requirements of Chapter 14 shall not apply to the design of masonry not laid in running bond in horizontally spanning walls.

14.2.3.9 Glass unit masonry — The prescriptive design requirements of Chapter 14 shall not apply to the design of glass unit masonry.

14.2.3.10 AAC masonry — The prescriptive design requirements of Chapter 14 shall not apply to the design of AAC masonry.

14.2.3.11 Concrete masonry — Concrete masonry, designed in accordance with Chapter 14, shall comply with one of the following:

(a) The minimum normalized web area of concrete masonry units, determined in accordance with ASTM C140, shall not be less than 187,500 mm^2/m^2, or
(b) The member shall be grouted solid.

14.2.3.12 Support — The provisions of Chapter 14 shall not apply to masonry vertically supported on wood construction.

14.3—Lateral support

14.3.1 Maximum l/t and h/t

Masonry partition walls without openings shall be laterally supported in either the horizontal or the vertical direction so that l/t or h/t does not exceed the values given in Table 14.1or

Table 14.2. It shall not be permitted to decrease the cross-section of the partition wall between supports unless permitted by Section 14.3.2.

14.3.2 Openings

Masonry partition walls with single or multiple openings shall be laterally supported in either the horizontal or vertical direction so that $1/t$ or h/t does not exceed the values given in Table 14.1 or

Table 14.2 divided by $\sqrt{W_T/W_S}$.

W_S is the dimension of the structural wall strip measured perpendicular to the span of the wall strip and perpendicular to the thickness as shown in Table 14.1. W_S is measured from the edge of the opening. W_S shall be no less than $3t$ on each side of each opening. Therefore, at walls with multiple openings, jambs shall be no less than $6t$ between openings. For design purposes, the effective W_S shall not be assumed to be greater than $6t$. At non-masonry lintels, the edge of the opening shall be considered the edge of the non-masonry lintel. W_S shall occur uninterrupted over the full span of the wall.

W_T is the dimension, parallel to W_S, from the center of the opening to the opposite end of W_S as shown in Table 14.1.

Where there are multiple openings perpendicular to W_S, W_T shall be measured from the center of a virtual opening that encompasses such openings. Masonry elements within the virtual opening must be designed in accordance with Chapter 8 or Chapter 9.

For walls with openings that span no more than 1200 mm, parallel to W_S, if W_S is no less than 1200 mm, then it shall be permitted to ignore the effect of those openings.

The span of openings, parallel to W_S, shall be limited so that the span divided by t does not exceed the values given in Table 14.1or

Table 14.2.

14.3.3 Cantilever walls

The ratio of height-to-nominal-thickness for cantilevered partition walls shall not exceed 6 for solid masonry or 4 for hollow masonry.

14.3.4 Support elements

Lateral support shall be provided by cross walls, pilasters, or structural frame members when the limiting distance is taken horizontally; or by floors, roofs acting as diaphragms, or structural frame members when the limiting distance is taken vertically.

14.4—Anchorage

14.4.1 General

Masonry partition walls shall be anchored in accordance with this section.

14.4.2 Intersecting walls

Masonry partition walls depending upon one another for lateral support shall be anchored or bonded at locations where they meet or intersect by one of the following methods:

14.4.2.1 Fifty percent of the units at the intersection shall be laid in an overlapping masonry bonding pattern, with alternate units having a bearing of at least 75.0 mm on the unit below.

14.4.2.2 Walls shall be anchored at their intersection at vertical intervals of not more than 400 mm with joint reinforcement or 6.4 mm mesh galvanized hardware cloth.

14.4.2.3 Other metal ties, joint reinforcement or anchors, if used, shall be spaced to provide equivalent area of anchorage to that required by Section 14.4.2.2.

14.5—Miscellaneous requirements

14.5.1 Chases and recesses

Masonry directly above chases or recesses wider than 300 mm shall be supported on lintels.

14.5.2 Lintels

The design of masonry lintels shall be in accordance with the provisions of Section 5.2.

14.5.3 Lap splices

Lap splices for bar reinforcement or joint reinforcement, required by Section 7.4.3.1 and located in masonry partition walls designed in accordance with this Chapter, shall be a minimum of $48d_b$.

TABLES AND FIGURES OF CHAPTER 14

Table 14.1: Maximum l/t[1] or h/t[1] for 0.240 kPa lateral load.[2]

Unit and Masonry Type	Mortar types			
	Portland cement/lime or mortar cement		Masonry cement or air entrained portland cement/lime	
	M or S	N	M or S	N
Ungrouted and partially grouted hollow units.[3]	26	24	22	18
Solid units and fully grouted hollow units.[3]	40	36	33	26

[1] *t* by definition is the nominal thickness of member

[2] See Section 14.2.3.2.

[3] For non-cantilevered walls laterally supported at both ends. See Section 14.3.3 for cantilevered walls.

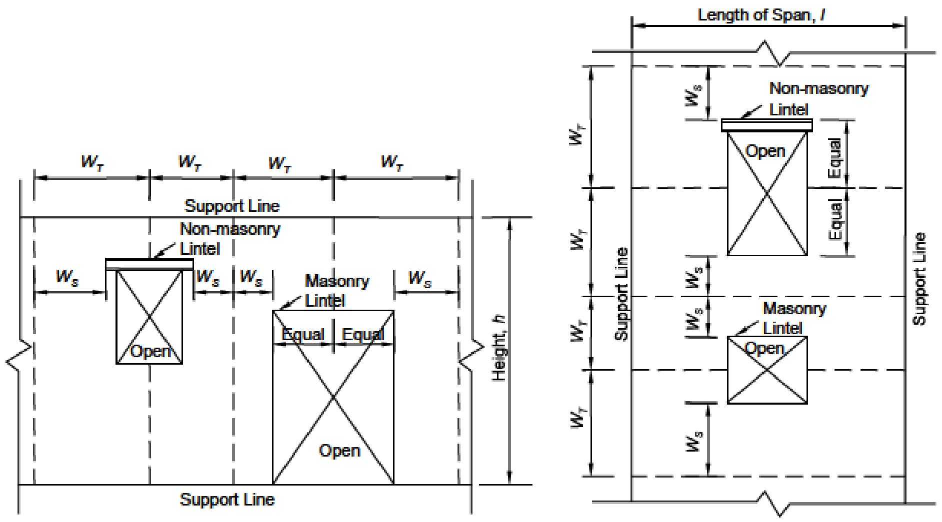
Table 14.2: Maximum l/t[1] or h/t[1] for 0.480 kPa lateral load.[2]

Unit and Masonry Type	Mortar types			
	Portland cement/lime or mortar cement		Masonry cement or air entrained portland cement/lime	
	M or S	N	M or S	N
Ungrouted and partially grouted hollow units.[3]	18	16	14	12
Solid units and fully grouted hollow units.[3]	28	24	22	18

[1] *t* by definition is the nominal thickness of member

[2] See Section 14.2.3.2.

[3] For non-cantilevered walls laterally supported at both ends. See Section 14.3.3 for cantilevered walls



W_S and W_T for Walls Spanning Vertically W_S and W_T for Walls Spanning Horizontally

Figure 14.1: Graphical representation of W_S and W_T

PART 5—APPENDICES

APPENDIX A—EMPIRICAL DESIGN OF MASONRY

A.1—General

A.1.1 Scope

This appendix provides requirements for empirical design of masonry.

A.1.1.1 The provisions of Part 1 and Part 2, excluding Part 1 Sections 1.2.1(c), 1.2.2, 4.1, 4.2 and 4.3, shall apply to empirical design, except as specifically stated in this Chapter.

A.1.1.2 Article 1.4 of TMS 602/ACI 530.1/ASCE 6 shall not apply to empirically designed masonry.

A.1.2 Limitations

A.1.2.1 Gravity Loads — The resultant of gravity loads shall be placed within the center third of the wall thickness and within the central area bounded by lines at one-third of each cross-sectional dimension of foundation piers.

A.1.2.2 Seismic — Empirical requirements shall not apply to the design of masonry for buildings, parts of buildings or other structures in Seismic Design Category D as defined in SBC 301, and shall not apply to the design of the seismic-force-resisting system for structures in Seismic Design Categories B or C.

A.1.2.3 Wind — Empirical requirements shall be permitted to be applied to the design of masonry elements defined by Table A.1, based on building height and basic wind speed that are applicable to the building.

A.1.2.4 Buildings and other structures in Risk Category IV — Empirical requirements shall not apply to the design of masonry for buildings, parts of buildings or other structures in Risk Category IV as defined in SBC 301.

A.1.2.5 Other horizontal loads — Empirical requirements shall not apply to structures resisting horizontal loads other than permitted wind or seismic loads or foundation walls as provided in Section A.6.3.

A.1.2.6 Glass unit masonry — The provisions of APPENDIX A shall not apply to glass unit masonry.

A.1.2.7 AAC masonry — The provisions of APPENDIX A shall not apply to AAC masonry.

A.1.2.8 Concrete masonry — Concrete masonry, designed in accordance with APPENDIX A, shall comply with one of the following:

(a) The minimum normalized web area of concrete masonry units, determined in accordance with ASTM C140, shall not be less than 187,500 mm^2/m^2, or

(b) The member shall be grouted solid.

A.1.2.9 Support — The provisions of Appendix A shall not apply to masonry vertically supported on wood construction.

A.1.2.10 Partition walls — The provisions of Appendix A shall not apply to partition walls.

A.2—Height

Buildings relying on masonry walls as part of their lateral-force-resisting system shall not exceed 10.70 m in height.

A.3—Lateral stability

A.3.1 Shear walls

Where the structure depends upon masonry walls for lateral stability, shear walls shall be provided parallel to the direction of the lateral forces resisted.

A.3.1.1 In each direction in which shear walls are required for lateral stability, shear walls shall be positioned in at least two separate planes parallel with the direction of the lateral force. The minimum cumulative length of shear walls provided along each plane shall be 0.2 multiplied by the long dimension of the building. Cumulative length of shear walls shall not include openings or any element whose length is less than one-half its height.

A.3.2 Shear walls shall be spaced so that the length-to-width ratio of each diaphragm transferring lateral forces to the shear walls does not exceed values given in Table A.3.

A.3.3 Roofs

The roof construction shall be designed so as not to impart out-of-plane lateral thrust to the walls under roof gravity load.

A.4—Compressive stress requirements

A.4.1 Calculations

Dead loads and live loads shall be in accordance with SBC 301, with such live load reductions as are permitted in SBC 301. Compressive stresses in masonry due to vertical dead plus live loads (excluding wind or seismic loads) shall be determined in accordance with the following:

(a) Stresses shall be calculated based on specified dimensions.
(b) Calculated compressive stresses for single wythe walls and for multiwythe composite masonry walls shall be determined by dividing the design load by the gross cross-sectional area of the member. The area of openings, chases, or recesses in walls shall not be included in the gross cross-sectional area of the wall.

A.4.2 Allowable compressive stresses

The compressive stresses in masonry shall not exceed the values given in Table A.4. In multiwythe walls, the allowable stresses shall be based on the weakest combination of the units and mortar used in each wythe.

A.5—Lateral support

A.5.1 Maximum l/t and h/t

Masonry walls without openings shall be laterally supported in either the horizontal or the vertical direction so that l/t or h/t does not exceed the values given in Table A.5.

Masonry walls with single or multiple openings shall be laterally supported in either the horizontal or vertical direction so that l/t or h/t does not exceed the values given in Table A.5 divided by $\sqrt{W_T/W_S}$

W_S is the dimension of the structural wall strip measured perpendicular to the span of the wall strip and perpendicular to the thickness as shown in Figure A.2. W_S is measured from the edge of the opening. W_S shall be no less than $3t$ on each side of each opening. Therefore, at walls with multiple openings, jambs shall be no less than $6t$ between openings. For design purposes, the effective W_S shall not be assumed to be greater than $6t$. At non-masonry lintels, the edge of the opening shall be considered the edge of the non-masonry lintel. W_S shall occur uninterrupted over the full span of the wall.

W_T is the dimension, parallel to W_S, from the center of the opening to the opposite end of W_S as shown in Figure A.2. Where there are multiple openings perpendicular to W_S, W_T shall be measured from the center of a virtual opening that encompasses such openings. Masonry elements within the virtual opening must be designed in accordance with Chapter 8 or Chapter 9.

For walls with openings that span no more than 1200 mm, parallel to W_S, if W_S is no less than 1200 mm, then it shall be permitted to ignore the effect of those openings.

The span of openings, parallel to W_S, shall be limited so that the span divided by t does not exceed the values given in Table A.5.

In addition to these limitations, lintels shall be designed for gravity loads in accordance with Section A.9.2.

In calculating the ratio for multiwythe walls, use the following thickness:

(1) The nominal wall thicknesses for solid walls and for hollow walls bonded with masonry headers (Section A.7.2).
(2) The sum of the nominal thicknesses of the wythes for non-composite walls connected with wall ties (Section A.7.3).

A.5.2 Cantilever walls

Except for parapets, the ratio of height-to-nominal thickness for cantilever walls shall not exceed 6 for solid masonry or 4 for hollow masonry. For parapets see Section A.6.4.

A.5.3 Support elements

Lateral support shall be provided by cross walls, pilasters, or structural frame members when the limiting distance is taken horizontally; or by floors, roofs acting as diaphragms, or structural frame members when the limiting distance is taken vertically.

A.6—THICKNESS OF MASONRY

A.6.1 General

Minimum thickness requirements shall be based on nominal dimensions of masonry.

A.6.2 Minimum thickness

A.6.2.1 Load-bearing walls — The minimum thickness of load-bearing walls of one story buildings shall be 150 mm. The minimum thickness of load-bearing walls of buildings more than one story high shall be 200 mm.

A.6.2.2 Rubble stone walls — The minimum thickness of rough, random, or coursed rubble stone walls shall be 400 mm.

A.6.2.3 Shear walls — The minimum thickness of masonry shear walls shall be 200 mm.

A.6.2.4 Foundation walls — The minimum thickness of foundation walls shall be 200 mm.

A.6.2.5 Foundation piers — The minimum thickness of foundation piers shall be 200 mm.

A.6.2.6 Parapet walls — The minimum thickness of parapet walls shall be 200 mm.

A.6.2.7 Partition walls — The minimum thickness of partition walls shall be 100 mm.

A.6.2.8 Change in thickness — Where walls of masonry of hollow units or masonry bonded hollow walls are decreased in thickness, a course or courses of solid masonry units or fully grouted hollow masonry units shall be interposed between the wall below and the thinner wall above, or special units or construction shall be used to transmit the loads from face shells or wythes above to those below.

A.6.3 Foundation walls

A.6.3.1 Foundation walls shall comply with the requirements of Table A.6, which are applicable when:

(a) the foundation wall does not exceed 2.45 m in height between lateral supports,
(b) the terrain surrounding foundation walls is graded to drain surface water away from foundation walls,
(c) backfill is drained to remove ground water away from foundation walls,
(d) lateral support is provided at the top of foundation walls prior to backfilling,
(e) the length of foundation walls between perpendicular masonry walls or pilasters is a maximum of 3 multiplied by the basement wall height,
(f) the backfill is granular and soil conditions in the area are non-expansive, and
(g) Masonry is laid in running bond using Type M or S mortar.

A.6.3.2 Where the requirements of Section A.6.3.1 are not met, foundation walls shall be designed in accordance with Part 1, Part 2, and Chapter 8 or Chapter 9.

A.6.4 Foundation piers

Design of foundation piers shall comply with Appendix A and the following:

(a) Length, measured perpendicular to its thickness, shall not exceed 3 times its thickness.
(b) Height shall be equal to or less than 4 times its thickness.

A.7—Bond

A.7.1 General

Wythes of multiple wythe masonry walls shall be bonded in accordance with the requirements of Section A.7.2, Section A.7.3, or Section A.7.4.

A.7.2 Bonding with masonry headers

A.7.2.1 Solid units — Where adjacent wythes of solid masonry walls are bonded by means of masonry headers, no less than 4 percent of the wall surface area of each face shall be composed of headers extending not less than 75 mm into each wythe. The distance between adjacent full-length headers shall not exceed 610 mm either vertically or horizontally. In multiwythe walls that are thicker than the length of a header, each wythe shall be connected to the adjacent wythe by adjacent headers that overlap a minimum of 75 mm.

A.7.2.2 Hollow units — Where two or more wythes are constructed using hollow units, the stretcher courses shall be bonded at vertical intervals not exceeding 860 mm by lapping at least 75 mm over the unit below, or by lapping at vertical intervals not exceeding 430 mm with units which are at least 50 percent greater in thickness than the units below.

A.7.3 Bonding with wall ties or joint reinforcement

A.7.3.1 Where adjacent wythes of masonry walls are bonded with wire size WD 5.0 wall ties or metal wire of equivalent stiffness embedded in the horizontal mortar joints, there shall be at least one metal tie for each 0.42 m^2 of wall area. The maximum vertical distance between ties shall not exceed 610 mm, and the maximum horizontal distance shall not exceed 910 mm. Rods or ties bent to rectangular shape shall be used with hollow masonry units laid with the cells vertical. In other walls, the ends of ties shall be bent to 90-degree angles to provide hooks no less than 50 mm long. Wall ties shall be without drips and shall be non-adjustable. Additional bonding ties shall be provided at openings, spaced not more than 0.90 m apart around the perimeter and within 300 mm of the opening.

A.7.3.2 Where adjacent wythes of masonry are bonded with prefabricated joint reinforcement, there shall be at least one cross wire serving as a tie for each 0.25 m^2 of wall area. The vertical spacing of the joint reinforcement shall not exceed 610 mm. Cross wires on prefabricated joint reinforcement shall be not smaller than wire size WD 4.0 and shall be without drips. The longitudinal wires shall be embedded in the mortar.

A.7.4 Natural or cast stone

A.7.4.1 Ashlar masonry — In ashlar masonry, uniformly distributed bonder units shall be provided to the extent of not less than 10 percent of the wall area. Such bonder units shall extend not less than 100 mm into the backing wall.

A.7.4.2 Rubble stone masonry — Rubble stone masonry 610 mm or less in thickness shall have bonder units with a maximum spacing of 0.90 m vertically and 0.90 m horizontally, and if the masonry is of greater thickness than 610 mm, shall have one bonder unit for each 0.56 m^2 of wall surface on both sides.

A.8—Anchorage

A.8.1 General

Masonry elements shall be anchored in accordance with this section.

A.8.2 Intersecting walls

Masonry walls depending upon one another for lateral support shall be anchored or bonded at locations where they meet or intersect by one of the following methods:

A.8.2.1 Fifty percent of the units at the intersection shall be laid in an overlapping masonry bonding pattern, with alternate units having a bearing of not less than 75 mm on the unit below.

A.8.2.2 Walls shall be anchored by steel connectors having a minimum section of 6.4 mm by 38 mm with ends bent up at least 51 mm, or with cross pins to form anchorage. Such anchors shall be at least 610 mm long and the maximum spacing shall be 1.22 m.

A.8.2.3 Walls shall be anchored by joint reinforcement spaced at a maximum distance of 200 mm. longitudinal wires of such reinforcement shall be at least wire size WD 4.0 and shall extend at least 760 mm in each direction at the intersection.

A.8.2.4 Other metal ties, joint reinforcement or anchors, if used, shall be spaced to provide equivalent area of anchorage to that required by Sections A.8.2.2 through A.8.2.4.

A.8.3 Floor and roof anchorage

Floor and roof diaphragms providing lateral support to masonry shall be connected to the masonry by one of the following methods:

A.8.3.1 Roof loading shall be determined by the provisions of Section 4.1.2 and, where net uplift occurs, uplift shall be resisted entirely by an anchorage system designed in accordance with the provisions of Sections 8.1 and 8.3 and, Sections 9.1 and 9.3.

A.8.3.2 Steel joists that are supported by masonry walls shall bear on and be connected to steel bearing plates. Maximum joist spacing shall be 1.83 m on center. Each bearing plate shall be anchored to the wall with a minimum of two 12.7 mm diameter bolts, or their equivalent. Where steel joists are parallel to the wall, anchors shall be located where joist bridging terminates at the wall and additional anchorage shall be provided to comply with Section A.8.3.3.

A.8.3.3 Roof and floor diaphragms shall be anchored to masonry walls with a minimum of 12.7 mm diameter bolts at a maximum spacing of 1.83 m on center or their equivalent.

A.8.3.4 Bolts and anchors required by Sections A.8.3.3 and A.8.3.4 shall comply with the following:

(a) Bolts and anchors at steel floor joists and floor diaphragms shall be embedded in the masonry at least 150 mm or shall comply with Section A.8.3.4(c).

(b) Bolts at steel roof joists and roof diaphragms shall be embedded in the masonry at least 380 mm or shall comply with Section A.8.3.4(c).

(c) In lieu of the embedment lengths listed in Sections A.8.3.4(a) and A.8.3.4(b), bolts shall be permitted to be hooked or welded to not less than 129 mm^2 of bond beam reinforcement placed not less than 150 mm below joist bearing or bottom of diaphragm.

A.8.4 Walls adjoining structural framing

Where walls are dependent upon the structural frame for lateral support, they shall be anchored to the structural members with metal anchors or otherwise keyed to the structural members. Metal anchors shall consist of 12.7-mm bolts spaced at 1.22 m on center embedded 100 mm into the masonry, or their equivalent area.

A.9—Miscellaneous requirements

A.9.1 Chases and recesses

Masonry directly above chases or recesses wider than 300 mm shall be supported on lintels.

A.9.2 Lintels

The design of masonry lintels shall be in accordance with the provisions of Section 5.2.

TABLES AND FIGURES OF APPENDIX A

Table A.1: Limitations based on building height and basic wind speed

<table>
<tr><th rowspan="2">Element Description</th><th rowspan="2">Building Height, m</th><th colspan="4">Basic Wind Speed, m/s (meter per second).[1]</th></tr>
<tr><th>Less than or equal 51</th><th>Over 51 and less than or equal to-54</th><th>Over 54 and less than or equal to 56</th><th>Over 56</th></tr>
<tr><td>Masonry elements that are part of the lateral-force-resisting system</td><td>11 and less</td><td colspan="3">Permitted</td><td>Not Permitted</td></tr>
<tr><td rowspan="4">Interior masonry loadbearing elements that are not part of the lateral-force-resisting system in buildings other than enclosed as defined by SBC 301</td><td>Over 55</td><td colspan="4">Not Permitted</td></tr>
<tr><td>Over 18 and less than or equal to 55</td><td>Permitted</td><td colspan="3">Not Permitted</td></tr>
<tr><td>Over 11 and less than or equal to 18</td><td colspan="2">Permitted</td><td colspan="2">Not Permitted</td></tr>
<tr><td>11 and less</td><td colspan="3">Permitted</td><td>Not Permitted</td></tr>
<tr><td rowspan="3">Exterior masonry elements that are not part of the lateral-force-resisting system</td><td>Over 55</td><td colspan="4">Not Permitted</td></tr>
<tr><td>Over 18 and less than or equal to 55</td><td>Permitted</td><td colspan="3">Not Permitted</td></tr>
<tr><td>Over 11 and less than or equal to 18</td><td colspan="2">Permitted</td><td colspan="2">Not Permitted</td></tr>
<tr><td>Exterior masonry elements</td><td>11 and less</td><td colspan="3">Permitted</td><td>Not Permitted</td></tr>
</table>

.[1] Basic wind speed as given in SBC 301

Table A.2: Checklist for use of Appendix A — Empirical Design of Masonry

<table>
<tr><td>1.</td><td colspan="3">Risk Category IV structures are not permitted to be designed using Appendix A.</td></tr>
<tr><td>2.</td><td colspan="3">Partitions are not permitted to be designed using Appendix A.</td></tr>
<tr><td rowspan="7">3.</td><td colspan="3">Use of empirical design is limited based on Seismic Design Category, as described in the following table.</td></tr>
<tr><td>Seismic Design Category</td><td>Participating Walls</td><td>Non-Participating Walls, except partition walls</td></tr>
<tr><td>A</td><td>Allowed by Appendix A</td><td>Allowed by Appendix A</td></tr>
<tr><td>B</td><td>Not Allowed</td><td>Allowed by Appendix A</td></tr>
<tr><td>C</td><td>Not Allowed</td><td>With prescriptive reinforcement per 7.4.3.1[1]</td></tr>
<tr><td>D</td><td>Not Allowed</td><td>Not Allowed</td></tr>
<tr><td colspan="3">[1] Lap splices are required to be designed and detailed in accordance with the requirements of Chapter 8 or Chapter 9.</td></tr>
<tr><td>4.</td><td colspan="3">Use of empirical design is limited based on wind speed at the project site, as described in Code A.1.2.3 and Code Table A.1.</td></tr>
<tr><td>5.</td><td colspan="3">If wind uplift on roofs result in net tension, empirical design is not permitted (A.8.3.1).</td></tr>
<tr><td>6.</td><td colspan="3">Loads used in the design of masonry must be listed on the design drawings (1.2.1(b)).</td></tr>
<tr><td>7.</td><td colspan="3">Details of anchorage to structural frames must be included in the design drawings (1.2.1(e)).</td></tr>
<tr><td>8.</td><td colspan="3">The design is required to include provisions for volume change (1.2.1(h)). The design drawings are required to include the locations and sizing of expansion, control, and isolation joints.</td></tr>
<tr><td>9.</td><td colspan="3">If walls are connected to structural frames, the connections and walls are required to be designed to resist the interconnecting forces and to accommodate deflections (4.4).

This provision requires a lateral load and uplift analysis for exterior walls that receive wind load and are supported by or are supporting a frame or roofing system.</td></tr>
<tr><td>10.</td><td colspan="3">Masonry not laid in running bond (for example, stack bond masonry) is required to have horizontal reinforcement (4.5).</td></tr>
<tr><td>11.</td><td colspan="3">A project quality assurance plan is required (3.1) with minimum requirements given in Table 3.1.</td></tr>
<tr><td>12.</td><td colspan="3">The resultant of gravity loads must be determined and assured to be located within certain limitations for walls and piers (A.1.2.1).</td></tr>
<tr><td>13.</td><td colspan="3">Ensure compliance of the design with prescriptive floor, roof, and wall-to-structural framing anchorage requirements, as well as other anchorage requirements (A.8.3 and A.8.4).</td></tr>
<tr><td>14.</td><td colspan="3">Type N mortar is not permitted for foundation walls (A.6.3.1(g)).</td></tr>
<tr><td>15.</td><td colspan="3">Design shear wall lengths, spacings, and orientations to meet the requirements of Code A.3.1.</td></tr>
</table>

Table A.3: Diaphragm length-to-width ratios

Floor or roof diaphragm construction	Maximum length-to-width ratio of diaphragm panel
Cast-in-place concrete	5:1
Precast concrete	4:1
Metal deck with concrete	3:1
Metal deck with no fill	2:1

Table A.4: Allowable compressive stresses for empirical design of masonry

Construction; compressive strength of masonry unit, gross area, MPa	Allowable compressive stresses.[1] based on gross cross-sectional area, MPa	
	Type M or S mortar	Type N mortar
Solid masonry of brick and other solid units of clay or shale; sand-lime or concrete brick:		
55.00 or greater	2.41	2.07
31.00	1.55	1.38
17.20	1.10	0.97
10.30	0.79	0.69
Grouted masonry of clay or shale; sand-lime or concrete:		
31.00 or greater	1.55	1.38
17.20	1.10	0.97
10.30	0.79	0.69
Solid masonry of solid concrete masonry units:		
20.70 or greater	1.55	1.38
13.80	1.10	0.97
8.30	0.79	0.69
Masonry of hollow load-bearing units of clay or shale.[2]:		
13.80 or greater	0.97	0.83
10.30	0.79	0.69
6.90	0.52	0.48
4.80	0.41	0.38
Masonry of hollow load-bearing concrete masonry units, up to and including 200 mm nominal thickness:		
13.80 or greater	0.97	0.83
10.30	0.79	0.69
6.90	0.52	0.48
4.80	0.41	0.38
Masonry of hollow load-bearing concrete masonry units, greater than 200 to 300 mm nominal thickness:		
13.80 or greater	0.86	0.76
10.30	0.72	0.62
6.90	0.45	0.41
4.80	0.38	0.35
Masonry of hollow load-bearing concrete masonry units, 300 mm nominal thickness and greater:		
13.80 or greater	0.79	0.69
10.30	0.66	0.59
6.90	0.41	0.38
4.80	0.35	0.31
Multiwythe non-composite walls.[2]:		
Solid units:		
17.20 or greater	1.10	0.97
10.30	0.79	0.69
Hollow units of clay or shale	0.52	0.48
Hollow units of concrete masonry of nominal thickness,		
up to and including 200 mm:	0.52	0.48
greater than 200-300 mm:	0.48	0.45
300 mm and greater:	0.41	0.38
Stone ashlar masonry:		
Granite	4.96	4.41
Limestone or marble	3.10	2.76
Sandstone or cast stone	2.48	2.21
Rubble stone masonry:		
Coursed, rough, or random	0.83	0.69

.[1] Linear interpolation shall be permitted for determining allowable stresses for masonry units having compressive strengths which are intermediate between those given in the table.

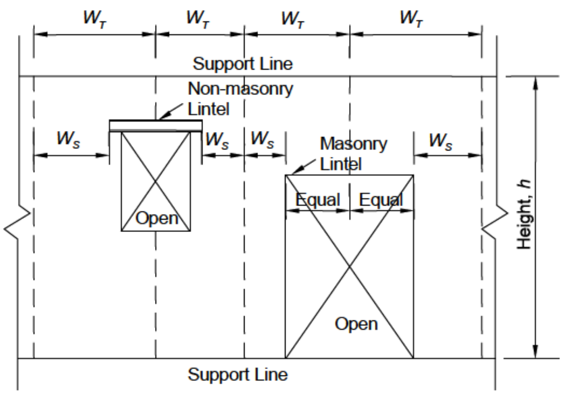
.[2] In non-composite walls, where floor and roof loads are carried upon one wythe, the gross cross-sectional area is that of the wythe under load; if both wythes are loaded, the gross cross-sectional area is that of the wall minus the area of the cavity between the wythes.

Table A.5: Diaphragm length-to-width ratios

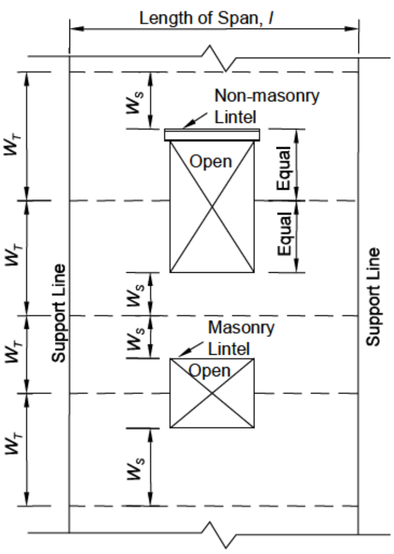
Construction	Maximum l/t or h/t
Load-bearing walls	
Solid units or fully grouted	20
Other than solid units or fully grouted	18
Non-load-bearing walls	
Exterior	18

Table A.6: Diaphragm length-to-width ratios

Wall construction	Nominal wall thickness, mm	Maximum depth of unbalanced backfill, m
Masonry of hollow units	200	1.50
	250	1.80
	300	2.10
Masonry of solid units	200	1.50
	250	2.10
	300	2.10
Fully grouted masonry	200	2.10
	250	2.45
	300	2.45

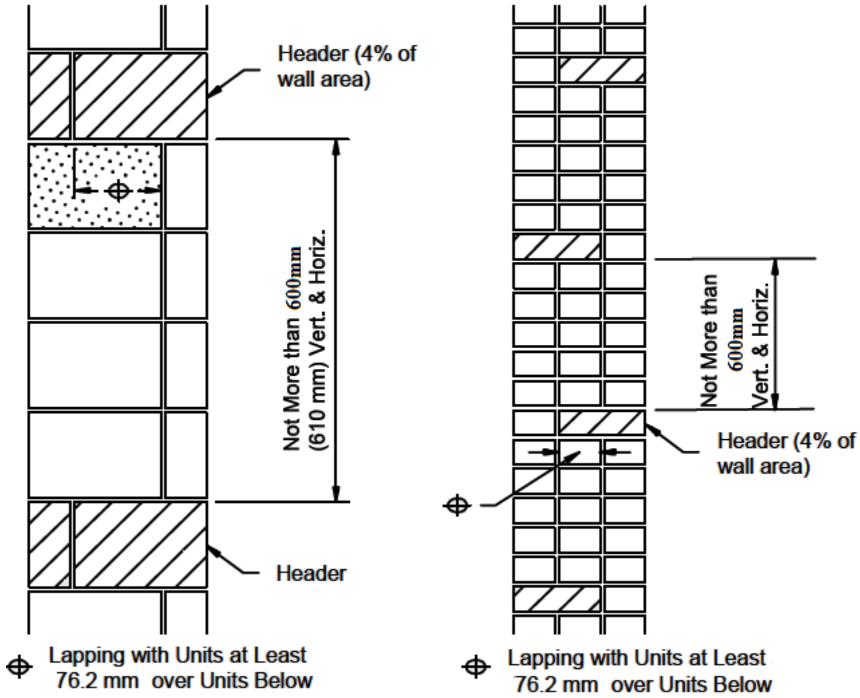


W_S and W_T for Walls Spanning Vertically



W_S and W_T for Walls Spanning Horizontally

Figure A.2: Graphical representation of W_S and W_T



a. Solid Units

b. Solid Units

APPENDIX B—DESIGN OF MASONRY INFILL

B.1—General

B.1.1 Scope

This chapter provides minimum requirements for the structural design of concrete masonry, clay masonry, and AAC masonry infills, either non-participating or participating. Infills shall comply with the requirements of Part 1, Part 2, excluding Sections 5.2, 5.3, 5.4, and 5.5, Section A1, and either Section B2 or B3.

B.1.2 Required strength

Required strength shall be determined in accordance with the strength design load combinations specified in SBC 301.

B.1.3 Design strength

Infills shall be proportioned so that the design strength equals or exceeds the required strength. Design strength is the nominal strength multiplied by the strength-reduction factor, ϕ, as specified in Section B.1.4.

B.1.4 Strength-reduction factors

The value of ϕ shall be taken as 0.60, and applied to the shear, flexure, and axial strength of a masonry infill panel.

B.1.5 Limitations

Partial infills and infills with openings shall not be considered as part of the lateral force-resisting system. Their effect on the bounding frame, however, shall be considered.

B.2—Non-participating infills

Non-participating infills shall comply with the requirements of Sections B.2.1 and B.2.2.

B.2.1 In-plane isolation joints for non-participating infills

B.2.1.1 In-plane isolation joints shall be designed between the infill and the sides and top of the bounding frame.

B.2.1.2 In-plane isolation joints shall be specified to be at least 9.5 mm wide in the plane of the infill, and shall be sized to accommodate the design displacements of the bounding frame.

B.2.1.3 In-plane isolation joints shall be free of mortar, debris, and other rigid materials, and shall be permitted to contain resilient material, provided that the compressibility of that material is considered in establishing the required size of the joint.

B.2.2 Design of non-participating infills for out-of-place loads

Connectors supporting non-participating infills against out-of-plane loads shall be designed to meet the requirements of Sections B.2.2.1 through B.2.2.4. The infill shall be designed to meet the requirements of Section B.2.2.5.

B.2.2.1 The connectors shall be attached to the bounding frame.

B.2.2.2 The connectors shall not transfer in-plane forces.

B.2.2.3 The connectors shall be designed to satisfy the requirements of SBC 301

B.2.2.4 The connectors shall be spaced at a maximum of 1.22 m along the supported perimeter of the infill.

B.2.2.5 The infill shall be designed to resist out-of-place bending between connectors in accordance with Section 9.2 for unreinforced concrete masonry or clay masonry infill, Section 11.2 for unreinforced AAC masonry infill, Section 9.3 for reinforced concrete masonry or clay masonry infill, or Section 11.3 for reinforced AAC masonry infill.

B.3—Participating infills

Participating infills shall comply with the requirements of Sections B.3.1 through B.3.6.

B.3.1 General

Infills with in-plane isolation joints not meeting the requirements of Section B.2.1 shall be considered as participating infills. For such infills the displacement shall be taken as the bounding frame displacement minus the specified width of the gap between the bounding column and infill.

B.3.1.1 The maximum ratio of the nominal vertical dimension to nominal thickness of participating infills shall not exceed 30.

B.3.1.2 Participating infills that are not constructed in contact with the bounding beam or slab adjacent to their upper edge shall be designed in accordance with Section B.3.1.2.1 or B.3.1.2.2.

B.3.1.2.1 Where the specified gap between the bounding beam or slab at the top of the infill is less than 9.5 mm or the gap is not sized to accommodate design displacements, the infill shall be designed in accordance with Sections B.3.4 and B.3.5, except that the calculated stiffness and strength of the infill shall be multiplied by a factor of 0.5.

B.3.1.2.2 If the gap between the infill and the overlying bounding beam or slab is sized such that in-plane forces cannot be transferred between the bounding beam or slab and the infill, the infill shall be considered a partial infill and shall comply with Section B.1.5.

B.3.2 In-plane connection requirements for participating infills

Mechanical connections between the infill and the bounding frame shall be permitted provided that they do not transfer in-plane forces between the infill and the bounding frame.

B.3.3 Out-of-plane connection requirements for participating infills

B.3.3.1 Participating infills shall be supported out-of-plane by connectors attached to the bounding frame.

B.3.3.2 Connectors providing out-of-plane support shall be designed to satisfy the requirements of SBC 301.

B.3.3.3 Connectors providing out-of-plane support shall be spaced at a maximum of 1.22 m along the supported perimeter of the infill.

B.3.4 Design of participating infills for in-plane forces

B.3.4.1 Unless the stiffness of the infill is obtained by a more comprehensive analysis, a participating infill shall be analyzed as an equivalent strut, capable of resisting compression only; whose width is calculated using Equation B-1; whose thickness is the specified thickness of the infill; and whose elastic modulus is the elastic modulus of the infill.

$$w_{inf} = \frac{0.3}{\lambda_{strut} cos\theta_{strut}} \qquad \text{Equation B-1}$$

Where:

$$\lambda_{strut} = \sqrt[4]{\frac{E_m t_{net\ inf} sin2\theta_{strut}}{4E_{bc} I_{bc} h_{inf}}} \qquad \text{Equation B-2a}$$

For the design of concrete masonry and clay masonry infill; and

$$\lambda_{strut} = \sqrt[4]{\frac{E_{AAC} t_{net\ inf} sin2\theta_{strut}}{4E_{bc} I_{bc} h_{inf}}} \qquad \text{Equation B-2b}$$

for the design of AAC masonry infill.

B.3.4.2 Design forces in equivalent struts, as defined in Section B.3.4.1, shall be determined from an elastic analysis of a braced frame including such equivalent struts.

B.3.4.3 $V_{n\ inf}$ shall be the smallest of (a), (b), and (c) for concrete masonry and clay masonry infill and (b), (d), and (e) for AAC masonry infill:

$$(150\text{ mm}) t_{net\ inf} f'_m \qquad \text{Equation B-3}$$

(d) the calculated horizontal component of the force in the equivalent strut at a horizontal racking displacement of 25 mm.

$$V_n/1.5 \qquad \text{Equation B-4}$$

where V_n is the smallest nominal shear strength from Section 9.2.6, calculated along a bed joint.

$$(150\text{ mm}) t_{net\ inf} f'_{AAC} \qquad \text{Equation B-5}$$

$$V_{nAAC}/1.5 \qquad \text{Equation B-6}$$

where V_{nAAC} is the smallest nominal shear strength from Section 11.2.5, calculated along a bed joint.

B.3.5 Design of frame elements with participating infills for in-plane loads

B.3.5.1 Design each frame member not in contact with an infill for shear, moment, and axial force not less than the results from the equivalent strut frame analysis.

B.3.5.2 Design each bounding column in contact with an infill for shear and moment equal to not less than 1.1 multiplied by the results from the equivalent strut frame analysis, and for axial force not less than the results from that analysis. In addition, increase the design shear at each end of the column by the horizontal component of the equivalent strut force acting on that end under design loads.

B.3.5.3 Design each beam or slab in contact with an infill for shear and moment equal to at least 1.1 multiplied by the results from the equivalent strut frame analysis, and for an axial force not less than the results from that analysis. In addition, increase the design shear at each end of the beam or slab by the vertical component of the equivalent strut force acting on that end under design loads.

B.3.6 Design of participating infills for out-of-plane forces

The nominal out-of-plane flexural capacity to resist out-of-plane forces of the infill per unit area shall be determined in accordance with Equation B-7a for concrete masonry and clay masonry and Equation B-7b for AAC masonry:

$$q_{n\,inf} = 729{,}000(f'_m)^{0.75} t_{inf}^2 \left(\frac{\alpha_{arch}}{l_{inf}^{\ 2.5}} + \frac{\beta_{arch}}{h_{inf}^{\ 2.5}}\right) \quad \text{Equation B-7a}$$

$$q_{n\,inf} = 729{,}000(f'_{AAC})^{0.75} t_{inf}^2 \left(\frac{\alpha_{arch}}{l_{inf}^{\ 2.5}} + \frac{\beta_{arch}}{h_{inf}^{\ 2.5}}\right) \quad \text{Equation B-7b}$$

Where:

$$\alpha_{arch} = \frac{1}{h_{inf}}(E_{bc}I_{bc}h_{inf}^2)^{0.25} < 50 \quad \text{Equation B-8}$$

$$\beta_{arch} = \frac{1}{l_{inf}}(E_{bb}I_{bb}l_{inf}^2)^{0.25} < 50 \quad \text{Equation B-9}$$

In Equation B-7, t_{inf} shall not be taken greater than $1/8\ h_{inf}$. When bounding columns of different cross-sectional properties are used on either side of the infill, average properties shall be used to calculate this capacity. When bounding beams of different cross-sectional properties are used above and below the infill, average properties shall be used to calculate this capacity. In the case of a single story frame, the cross-sectional properties of the bounding beam above the infill shall be used to calculate this capacity. When a side gap is present, α_{arch} shall be taken as zero. When a top gap is present, β_{arch} shall be taken as zero.

APPENDIX C—LIMIT DESIGN METHOD

C.0—General

The limit design method shall be permitted to be applied to a line of lateral load resistance consisting of special reinforced masonry shear walls that are designed per the strength design provisions of Chapter 9, except that the provisions of Section 9.3.3.5 and Section 9.3.6.5 shall not apply.

C.1—Yield mechanism

It shall be permitted to use limit analysis to determine the controlling yield mechanism and its corresponding base-shear strength, V_{lim}, for a line of lateral load resistance, provided that (a) through (e) are satisfied:

(a) The relative magnitude of lateral seismic forces applied at each floor level shall correspond to the loading condition producing the maximum base shear at the line of resistance in accordance with analytical procedures permitted in Section 12.6 of SBC 301.

(b) In the investigation of potential yield mechanisms induced by seismic loading, plastic hinges shall be considered to form at the faces of joints and at the interfaces between masonry components and the foundation.

(c) The axial forces associated with Load Combination 7 of Section 2.3.2 of SBC 301 shall be used when determining the strength of plastic hinges, except that axial loads due to horizontal seismic forces shall be permitted to be neglected.

(d) The strength assigned to plastic hinges shall be based on the nominal flexural strength, M_n, but shall not exceed the moment associated with one-half of the nominal shear strength, V_n, calculated using MSJC Section 9.3.4.1.2.

(e) At locations other than the plastic hinges identified in C.1(b), moments shall not exceed the strengths assigned in C.1(d) using the assumptions of C.1(c).

C.2—Mechanism strength

The yield mechanism associated with the limiting base-shear strength, V_{lim} shall satisfy the following:

$$\phi V_{lim} \geq V_{ub} \qquad \text{Equation C-1}$$

The value of ϕ assigned to the mechanism strength shall be taken as 0.8. The base-shear demand, V_{ub}, shall be determined from analytical procedures permitted in Section 12.6 of SBC 301.

C.3—Mechanism deformation

The rotational deformation demand on plastic hinges shall be determined by imposing the design displacement, δ_u, at the roof level of the yield mechanism. The rotational deformation capacity of plastic hinges shall satisfy C.3.1 to C.3.3.

C.3.1 The rotational deformation capacity of plastic hinges shall be taken as $0.5 l_w \varepsilon_{mu}/c$. The value of c shall be calculated for the P_u corresponding to Load Combination 5 of Section 2.3.2 of SBC 301.

C.3.2 The angular deformation capacity of masonry components whose plastic hinge strengths are limited by shear as specified in C.1 (d), shall be taken as $1/400$. The angular deformation capacity shall be permitted to be taken as $1/200$ for masonry components satisfying the following requirements:

(a) The areas of transverse and longitudinal reinforcement shall each not be less than 0.001 multiplied by the gross cross-sectional area of the component, using specified dimensions;

(b) Spacing of transverse and longitudinal reinforcement shall not exceed the smallest of 610 mm, $l_w/2$, and $h_w/2$.

(c) Reinforcement ending at a free edge of masonry shall be anchored around perpendicular reinforcing bars with a standard hook.

C.3.3 The P_u corresponding to load combination 5 of Section 2.3.2 of SBC 301 shall not exceed a compressive stress of $0.3 f'_m A_g$ at plastic hinges in the controlling mechanism.

APPENDIX D—MASONRY FIREPLACES

D.1—General.

The construction of masonry fireplaces, consisting of concrete or masonry, shall be in accordance with this Appendix.

D.2—Fireplace drawings.

The construction documents shall describe in sufficient detail the location, size and construction of masonry fireplaces. The thickness and characteristics of materials and the clearances from walls, partitions and ceilings shall be indicated.

D.3—Footings and foundations.

Footings for masonry fireplaces and their chimneys shall be constructed of concrete or solid masonry at least 300 mm thick and shall extend at least 150 mm beyond the face of the fireplace or foundation wall on all sides. Footings shall be founded on natural undisturbed earth or engineered fill below frost depth. In areas not subjected to freezing, footings shall be at least 300 mm below finished grade.

D.3.1 Ash dump cleanout. Cleanout openings, located within foundation walls below fireboxes, when provided, shall be equipped with ferrous metal or masonry doors and frames constructed to remain tightly closed, except when in use. Cleanouts shall be accessible and located so that ash removal will not create a hazard to combustible materials.

D.4—Seismic reinforcement

In structures assigned to Seismic Design Category A or B, seismic reinforcement is not required. In structures assigned to Seismic Design Category C or D, masonry fireplaces shall be reinforced and anchored in accordance with Sections D.4.1, D.4.2 and D.5.

D.4.1 Vertical reinforcing. For fireplaces with chimneys up to 1000 mm wide, four Dia 12 continuous vertical bars, anchored in the foundation, shall be placed in the concrete between wythes of solid masonry or within the cells of hollow unit masonry and grouted in accordance with Article 202 of TMS 602/ACI530.1/ASCE6. For fireplaces with chimneys greater than 1000 mm wide, two additional Dia 12 vertical bars shall be provided for each additional 1000 mm in width or fraction thereof.

D.4.2 Horizontal reinforcing. Vertical reinforcement shall be placed enclosed within 6 mm ties or other reinforcing of equivalent net cross-sectional area, spaced not to exceed 450 mm on center in concrete; or placed in the bed joints of unit masonry at a minimum of every 450 mm of vertical height. Two such ties shall be provided at each bend in the vertical bars.

D.5—Seismic anchorage

Masonry fireplaces and foundations shall be anchored at each floor, ceiling or roof line more than 1800 mm above grade with two 5 mm by 25 mm straps embedded a minimum of 300 mm into the chimney. Straps shall be hooked around the outer bars and extend 150 mm beyond the bend. Each strap shall be fastened to a minimum of four floor joists with two 13 mm bolts.

Exception: Seismic anchorage is not required for the following:

(1) In structures assigned to Seismic Design Category A or B.

(2) Where the masonry fireplace is constructed completely within the exterior walls.

D.6—Firebox walls

Masonry fireboxes shall be constructed of solid masonry units, hollow masonry units grouted solid, stone or concrete. When a lining of firebrick at least 50 mm in thickness or other approved lining is provided, the minimum thickness of back and sidewalls shall each be 200 mm of solid masonry, including the lining. The width of joints between firebricks shall be not greater than 6 mm. When no lining is provided, the total minimum thickness of back and sidewalls shall be 250 mm of solid masonry. Firebrick shall conform to ASTM C 27 or ASTM C 1261 and shall be laid with medium-duty refractory mortar conforming to ASTM C 199.

D.6.1 Steel fireplace units. Steel fireplace units are permitted to be installed with solid masonry to form a masonry fireplace provided they are installed according to either the requirements of their listing or the requirements of this section. Steel fireplace units incorporating a steel firebox lining shall be constructed with steel not less than 6 mm in thickness, and an air-circulating chamber which is ducted to the interior of the building. The firebox lining shall be encased with solid masonry to provide a total thickness at the back and sides of not less than 200 mm, of which not less than 100 mm shall be of solid masonry or concrete. Circulating air ducts employed with steel fireplace units shall be constructed of metal or masonry.

D.7—Firebox dimensions

The firebox of a concrete or masonry fireplace shall have a minimum depth of 500 mm. The throat shall be not less than 200 mm above the fireplace opening. The throat opening shall not be less than 100 mm in depth. The cross-sectional area of the passageway above the firebox, including the throat, damper and smoke chamber, shall be not less than the cross-sectional area of the flue.

Exception: Rumford fireplaces shall be permitted provided that the depth of the fireplace is not less than 300 mm and at least one-third of the width of the fireplace opening, and the throat is not less than 300 mm above the lintel, and at least 1/20, the cross-sectional area of the fireplace opening.

D.8—Lintel and throat

Masonry over a fireplace opening shall be supported by a lintel of noncombustible material. The minimum required bearing length on each end of the fireplace opening shall be 100 mm. The fireplace throat or damper shall be located not less than 200 mm above the top of the fireplace opening.

D.8.1 Damper. Masonry fireplaces shall be equipped with a ferrous metal damper located not less than 200 mm above the top of the fireplace opening. Dampers shall be installed in the fireplace or at the top of the flue venting the fireplace, and shall be operable from the room containing the fireplace. Damper controls shall be permitted to be located in the fireplace.

D.9—Smoke chamber walls

Smoke chamber walls shall be constructed of solid masonry units, hollow masonry units grouted solid, stone or concrete. The total minimum thickness of front, back and sidewalls shall be 200 mm of solid masonry. The inside surface shall be parged smooth with refractory mortar conforming to ASTM C 199. When a lining of firebrick not less than 50 mm thick, or a lining of vitrified clay not less than 16 mm thick, is provided, the total minimum thickness of front, back and sidewalls shall be 150 mm of solid masonry, including the lining. Firebrick shall conform to ASTM C 1261 and shall be laid with refractory mortar conforming to ASTM C 199. Vitrified clay linings shall conform to ASTM C 315.

D.9.1 Smoke chamber dimensions. The inside height of the smoke chamber from the fireplace throat to the beginning of the flue shall be not greater than the inside width of the fireplace opening. The inside surface of the smoke chamber shall not be inclined more than 45 degrees (0.76 rad) from vertical when prefabricated smoke chamber linings are used or when the smoke chamber walls are rolled or sloped rather than corbeled. When the inside surface of the smoke chamber is formed by corbeled masonry, the walls shall not be corbeled more than 30 degrees (0.52 rad) from vertical.

D.10—Hearth and hearth extension

Masonry fireplace hearths and hearth extensions shall be constructed of concrete or masonry, supported by noncombustible materials, and reinforced to carry their own weight and all imposed loads. No combustible material shall remain against the underside of hearths or hearth extensions after construction.

D.10.1 Hearth thickness. The minimum thickness of fireplace hearths shall be 100 mm.

D.10.2 Hearth extension thickness. The minimum thickness of hearth extensions shall be 50 mm.

Exception: When the bottom of the firebox opening is raised not less than 200 mm above the top of the hearth extension, a hearth extension of not less than 10 mm brick, concrete, stone, tile or other approved noncombustible material is permitted.

D.11—Hearth extension dimensions

Hearth extensions shall extend not less than 400 mm in front of, and not less than 200 mm beyond, each side of the fireplace opening. Where the fireplace opening is 0.6 m^2 or larger, the hearth extension shall extend not less than 500 mm in front of, and not less than 300 mm beyond, each side of the fireplace opening.

D.12—Fireplace clearance

Any portion of a masonry fireplace located in the interior of a building or within the exterior wall of a building shall have a clearance to combustibles of not less than 50 mm from the front faces and sides of masonry fireplaces and not less than 100 mm from the back faces of masonry fireplaces. The airspace shall not be filled, except to provide fire blocking in accordance with Section D.13.

Exceptions:

(1) Masonry fireplaces listed and labeled for use in contact with combustibles in accordance with UL 127 and installed in accordance with the manufacturer's instructions are permitted to have combustible material in contact with their exterior surfaces.

(2) When masonry fireplaces are constructed as part of masonry or concrete walls, combustible materials shall not be in contact with the masonry or concrete walls less than 300 mm from the inside surface of the nearest firebox lining.

(3) Exposed combustible trim and the edges of sheathing materials, such as wood siding, flooring and drywall, are permitted to abut the masonry fireplace sidewalls and hearth extension, in accordance with Figure D.1, provided such combustible trim or sheathing is not less than 300 mm from the inside surface of the nearest firebox lining.

(4) Exposed combustible mantels or trim is permitted to be placed directly on the masonry fireplace front surrounding the fireplace opening, provided such combustible materials shall not be placed within 150 mm of a fireplace opening. Combustible material directly above and within 300 mm of the fireplace opening shall not project more than 3 mm for each 25 mm distance from such opening. Combustible materials located along the sides of the fireplace opening that project more than 38 mm from the face of the fireplace shall have an additional clearance equal to the projection.

D.13—Fireplace fire blocking

All spaces between fireplaces and floors and ceilings through which fireplaces pass shall be fire blocked with noncombustible material securely fastened in place. The fire blocking of spaces between wood joists, beams or headers shall be to a depth of 25 mm and shall only be placed on strips of metal or metal lath laid across the spaces between combustible material and the chimney.

D.14—Exterior air

Factory-built or masonry fireplaces covered in this section shall be equipped with an exterior air supply to ensure proper fuel combustion unless the room is mechanically ventilated and controlled so that the indoor pressure is neutral or positive.

D.14.1 Factory-built fireplaces — Exterior combustion air ducts for factory-built fireplaces shall be listed components of the fireplace, and installed according to the fireplace manufacturer's instructions.

D.14.2 Masonry fireplaces — Listed combustion air ducts for masonry fireplaces shall be installed according to the terms of their listing and manufacturer's instructions.

D.14.3 Exterior air intake — The exterior air intake shall be capable of providing all combustion air from the exterior of the dwelling. The exterior air intake shall not be located within a garage, attic, basement or crawl space of the dwelling nor shall the air intake be located at an elevation higher than the firebox. The exterior air intake shall be covered with a corrosion-resistant screen of 6 mm mesh.

D.14.4 Clearance — Unlisted combustion air ducts shall be installed with a minimum 25 mm clearance to combustibles for all parts of the duct within 1500 mm of the duct outlet.

D.14.5 Passageway — The combustion air passageway shall be not less than 3870 mm^2 and not more than 0.035 m^2, except that combustion air systems for listed fireplaces or for fireplaces tested for emissions shall be constructed according to the fireplace manufacturer's instructions.

D.14.6 Outlet — The exterior air outlet is permitted to be located in the back or sides of the firebox chamber or within 600 mm of the firebox opening on or near the floor. The outlet shall be closable and designed to prevent burning material from dropping into concealed combustible spaces.

TABLES AND FIGURES OF APPENDIX D

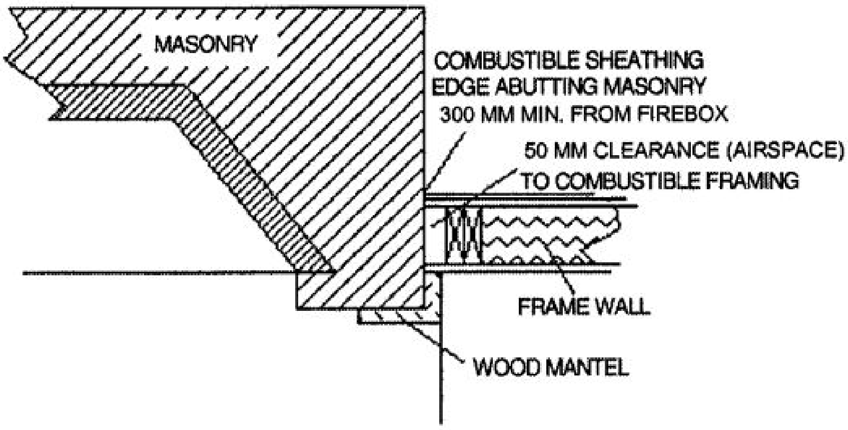


Figure D.1: Illustration of exception to fireplace clearance provision

APPENDIX E—MASONRY HEATERS

E.1—Definition

A masonry heater is a heating appliance constructed of concrete or solid masonry, hereinafter referred to as "masonry", which is designed to absorb and store heat from a solid fuel fire built in the firebox by routing the exhaust gases through internal heat exchange channels in which the flow path downstream of the firebox may include flow in a horizontal or downward direction before entering the chimney and which delivers heat by radiation from the masonry surface of the heater.

E.2—Installation

Masonry heaters shall be installed in accordance with this Appendix and comply with one of the following:

(1) Masonry heaters shall comply with the requirements of ASTM E 1602.

(2) Masonry heaters shall be listed and labeled in accordance with UL 1482 or EN 15250 and installed in accordance with the manufacturer's instructions.

E.3—Footings and foundation

The firebox floor of a masonry heater shall be a minimum thickness of 100 mm of noncombustible material and be supported on a noncombustible footing and foundation in accordance with Section F3.

E.4—Seismic reinforcing

In structures assigned to Seismic Design Category D, E or F, masonry heaters shall be anchored to the masonry foundation in accordance with Section F3. Seismic reinforcing shall not be required within the body of a masonry heater with a height that is equal to or less than 3.5 times its body width and where the masonry chimney serving the heater is not supported by the body of the heater. Where the masonry chimney shares a common wall with the facing of the masonry heater, the chimney portion of the structure shall be reinforced in accordance with Appendix F.

E.5—Masonry heater clearance

Combustible materials shall not be placed within 900 mm or the distance of the allowed reduction method from the outside surface of a masonry heater in accordance with NFPA 211, Section 12.6, and the required space between the heater and combustible material shall be fully vented to permit the free flow of air around all heater surfaces.

Exceptions:

(1) Where the masonry heater wall thickness is at least 200 mm of solid masonry and the wall thickness of the heat exchange channels is not less than 125 mm of solid masonry, combustible materials shall not be placed within 100 mm of the outside surface of a masonry heater. A clearance of not less than 200 mm shall be provided between the gas-tight capping slab of the heater and a combustible ceiling.

(2) Masonry heaters listed and labeled in accordance with UL 1482 or EN 15250 and installed in accordance with the manufacturer's instructions.

APPENDIX F—MASONRY CHIMNEYS

F.1—General

The construction of masonry chimneys consisting of solid masonry units, hollow masonry units grouted solid, stone or concrete shall be in accordance with this Appendix.

F.2—Footings and foundations

Footings for masonry chimneys shall be constructed of concrete or solid masonry not less than 300 mm thick and shall extend at least 150 mm beyond the face of the foundation or support wall on all sides. Footings shall be founded on natural undisturbed earth or engineered fill below frost depth. In areas not subjected to freezing, footings shall be not less than 300 mm below finished grade.

F.3—Seismic reinforcement

In structures assigned to Seismic Design Category A or B, seismic reinforcement is not required. In structures assigned to Seismic Design Category C or D, masonry chimneys shall be reinforced and anchored in accordance with Sections F.3.1, F.3.2 and F.4.

F.3.1 Vertical reinforcement. For chimneys up to 1000 mm wide, four Dia 12 continuous vertical bars anchored in the foundation shall be placed in the concrete between wythes of solid masonry or within the cells of hollow unit masonry and grouted in accordance with Article 2.2 of TMS 602/ACI530.1/ASCE6. Grout shall be prevented from bonding with the flue liner so that the flue liner is free to move with thermal expansion. For chimneys greater than 1000 mm wide, two additional Dia 12 vertical bars shall be provided for each additional 1000 mm in width or fraction thereof.

F.3.2 Horizontal reinforcement. Vertical reinforcement shall be placed enclosed within 6 mm ties, or other reinforcing of equivalent net cross-sectional area, spaced not to exceed 450 mm on center in concrete, or placed in the bed joints of unit masonry, at not less than every 450 mm of vertical height. Two such ties shall be provided at each bend in the vertical bars.

F.4—Seismic anchorage

Masonry chimneys and foundations shall be anchored at each floor, ceiling or roof line more than 1800 mm above grade with two 5 mm by 25 mm straps embedded not less than 300 mm into the chimney. Straps shall be hooked around the outer bars and extend 150 mm beyond the bend. Each strap shall be fastened to not less than four floor joists with two 13 mm bolts.

Exception: Seismic anchorage is not required for the following;

(1) In structures assigned to Seismic Design Category A or B.

(2) Where the masonry fireplace is constructed completely within the exterior walls.

F.5—Corbeling

Masonry chimneys shall not be corbeled more than half of the chimney's wall thickness from a wall or foundation, nor shall a chimney be corbeled from a wall or foundation that is less than 300 mm in thickness unless it projects equally on each side of the wall, except that on the second story of a two-story dwelling, corbeling of chimneys on the exterior of the enclosing walls is permitted to equal the wall thickness. The projection of a single course shall not exceed one-half the unit height or one-third of the unit bed depth, whichever is less.

F.6—Changes in dimension

The chimney wall or chimney flue lining shall not change in size or shape within 150 mm above or below where the chimney passes through floor components, ceiling components or roof components.

F.7—Offsets

Where a masonry chimney is constructed with a fireclay flue liner surrounded by one wythe of masonry, the maximum offset shall be such that the centerline of the flue above the offset does not extend beyond the center of the chimney wall below the offset. Where the chimney offset is supported by masonry below the offset in an approved manner, the maximum offset limitations shall not apply. Each individual corbeled masonry course of the offset shall not exceed the projection limitations specified in Section F5.

F.8—Additional load

Chimneys shall not support loads other than their own weight unless they are designed and constructed to support the additional load. Masonry chimneys are permitted to be constructed as part of the masonry walls or concrete walls of the building.

F.9—Termination

Chimneys shall extend not less than 600 mm higher than any portion of the building within 3000 mm, but shall not be less than 900 mm above the highest point where the chimney passes through the roof.

F.9.1 Chimney caps. Masonry chimneys shall have a concrete, metal or stone cap, sloped to shed water, a drip edge and a caulked bond break around any flue liners in accordance with ASTM C 1283.

F.9.2 Spark arrestors. Where a spark arrestor is installed on a masonry chimney, the spark arrestor shall meet all of the following requirements:

(1) The net free area of the arrestor shall be not less than four times the net free area of the outlet of the chimney flue it serves.

(2) The arrestor screen shall have heat and corrosion resistance equivalent to 19-gage galvanized steel or 24-gage stainless steel.

(3) Openings shall not permit the passage of spheres having a diameter greater than 13 mm nor block the passage of spheres having a diameter less than 10 mm.

(4) The spark arrestor shall be accessible for cleaning and the screen or chimney cap shall be removable to allow for cleaning of the chimney flue.

F.9.3 Rain caps. Where a masonry or metal rain cap is installed on a masonry chimney, the net free area under the cap shall be not less than four times the net free area of the outlet of the chimney flue it serves.

F.10—Wall thickness

Masonry chimney walls shall be constructed of concrete, solid masonry units or hollow masonry units grouted solid with not less than 100 mm nominal thickness.

F.10.1 Masonry veneer chimneys. Where masonry is used as veneer for a framed chimney, through flashing and weep holes shall be provided as required by CHAPTER 12.

F.11—Flue lining (material)

Masonry chimneys shall be lined. The lining material shall be appropriate for the type of appliance connected, according to the terms of the appliance listing and the manufacturer's instructions.

F.11.1 Residential-type appliances (general). Flue lining systems shall comply with one of the following:

(1) Clay flue lining complying with the requirements of ASTM C 315.

(2) Listed chimney lining systems complying with UL 1777.

(3) Factory-built chimneys or chimney units listed for installation within masonry chimneys.

(4) Other approved materials that will resist corrosion, erosion, softening or cracking from flue gases and condensate at temperatures up to 982°C.

F.11.1.1 Flue linings for specific appliances. Flue linings other than those covered in Section F.11.1 intended for use with specific appliances shall comply with Sections F.11.1.2 through F.11.1.4 and Sections F.11.2 and F.11.3.

F.11.1.2 Gas appliances. Flue lining systems for gas appliances shall be in accordance with the International Fuel Gas Code.

F.11.1.3 Pellet fuel-burning appliances. Flue lining and vent systems for use in masonry chimneys with pellet fuel-burning appliances shall be limited to flue lining systems complying with Section F.11.1 and pellet vents listed for installation within masonry chimneys (see Section F.11.1.5 for marking).

F.11.1.4 Oil-fired appliances approved for use with L-vent. Flue lining and vent systems for use in masonry chimneys with oil-fired appliances approved for use with Type L vent shall be limited to flue lining systems complying with Section F.11.1 and listed chimney liners complying with UL 641 (see Section F.11.1.5 for marking).

F.11.1.5 Notice of usage. When a flue is relined with a material not complying with Section F.11.1, the chimney shall be plainly and permanently identified by a label attached to a wall, ceiling or other conspicuous location adjacent to where the connector enters the chimney. The label shall include the following message or equivalent language: "This chimney is for use only with (type or category of appliance) that burns (type of fuel). Do not connect other types of appliances."

F.11.2 Concrete and masonry chimneys for medium-heat appliances.

F.11.2.1 General. Concrete and masonry chimneys for medium-heat appliances shall comply with Sections F1 through F5.

F.11.2.2 Construction. Chimneys for medium-heat appliances shall be constructed of solid masonry units or of concrete with walls not less than 200 mm thick, or with stone masonry not less than 300 mm thick

F.11.2.3 Lining. Concrete and masonry chimneys shall be lined with an approved medium-duty refractory brick not less than 110 mm thick laid on the 110 mm in an approved medium-duty refractory mortar. The lining shall start 600 mm or more below the lowest chimney connector entrance. Chimneys terminating 7500 mm or less above a chimney connector entrance shall be lined to the top.

F.11.2.4 Multiple passageway. Concrete and masonry chimneys containing more than one passageway shall have the liners separated by a minimum 100 mm concrete or solid masonry wall.

F.11.2.5 Termination height. Concrete and masonry chimneys for medium-heat appliances shall extend not less than 3000 mm higher than any portion of any building within 7500 mm.

F.11.2.6 Clearance. A minimum clearance of 100 mm shall be provided between the exterior surfaces of a concrete or masonry chimney for medium-heat appliances and combustible material.

F.11.3 Concrete and masonry chimneys for high-heat appliances.

F.11.3.1 General. Concrete and masonry chimneys for high-heat appliances shall comply with Sections F1 through F5.

F.11.3.2 Construction. Chimneys for high-heat appliances shall be constructed with double walls of solid masonry units or of concrete, each wall to be not less than 200 mm thick with a minimum air-space of 50 mm between the walls.

F.11.3.3 Lining. The inside of the interior wall shall be lined with an approved high-duty refractory brick, not less than 110 mm thick laid on the 110 mm in an approved high-duty refractory mortar. The lining shall start at the base of the chimney and extend continuously to the top.

F.11.3.4 Termination height. Concrete and masonry chimneys for high-heat appliances shall extend not less than 6100 mm higher than any portion of any building within 15200 mm.

F.11.3.5 Clearance. Concrete and masonry chimneys for high-heat appliances shall have approved clearance from buildings and structures to prevent overheating combustible materials, permit inspection and maintenance operations on the chimney and prevent danger of burns to persons.

F.12—Clay flue lining (installation)

Clay flue liners shall be installed in accordance with ASTM C 1283 and extend from a point not less than 203 mm below the lowest inlet or, in the case of fireplaces, from the top of the smoke chamber to a point above the enclosing walls. The lining shall be carried up vertically, with a maximum slope no greater than 30 degrees (0.52 rad) from the vertical.

Clay flue liners shall be laid in medium-duty non water-soluble refractory mortar conforming to ASTM C 199 with tight mortar joints left smooth on the inside and installed to maintain an airspace or insulation not to exceed the thickness of the flue liner separating the flue liners from the interior face of the chimney masonry walls. Flue lining shall be supported on all sides. Only enough mortar shall be placed to make the joint and hold the liners in position.

F.13—Additional requirements

F.13.1 Listed materials. Listed materials used as flue linings shall be installed in accordance with the terms of their listings and the manufacturer's instructions.

F.13.2 Space around lining. The space surrounding a chimney lining system or vent installed within a masonry chimney shall not be used to vent any other appliance.

Exception: This shall not prevent the installation of a separate flue lining in accordance with the manufacturer's instructions.

F.14—Multiple flues

When two or more flues are located in the same chimney, masonry wythes shall be built between adjacent flue linings. The masonry wythes shall be at least 100 mm thick and bonded into the walls of the chimney.

Exception: When venting only one appliance, two flues are permitted to adjoin each other in the same chimney with only the flue lining separation between them. The joints of the adjacent flue linings shall be staggered not less than 100 mm.

F.15—Flue area (appliance)

Chimney flues shall not be smaller in area than the area of the connector from the appliance. Chimney flues connected to more than one appliance shall be not less than the area of the largest connector plus 50 percent of the areas of additional chimney connectors.

Exceptions:

(1) Chimney flues serving oil-fired appliances sized in accordance with NFPA 31.

(2) Chimney flues serving gas-fired appliances sized in accordance with the International Fuel Gas Code.

F.16—Flue area (masonry fireplace).

Flue sizing for chimneys serving fireplaces shall be in accordance with Section F.16.1 or F.16.2.

F.16.1 Minimum area. Round chimney flues shall have a minimum net cross-sectional area of not less than 1/12 of the fireplace opening. Square chimney flues shall have a minimum net cross-sectional area of not less than 1/10 of the fireplace opening. Rectangular chimney flues with an aspect ratio less than 2 to 1 shall have a minimum net cross-sectional area of not less than 1/10 of the fireplace opening. Rectangular chimney flues with an aspect ratio of 2 to 1 or more shall have a minimum net cross-sectional area of not less than 1/8 of the fireplace opening.

F.16.2 Determination of minimum area. The minimum net cross-sectional area of the flue shall be determined in accordance with Figure F.1. A flue size providing not less than the equivalent net cross-sectional area shall be used. Cross-sectional areas of clay flue linings are as provided in Table F.1 and Table F.2 or as provided by the manufacturer or as measured in the field. The height of the chimney shall be measured from the firebox floor to the top of the chimney flue.

F.17—Inlet

Inlets to masonry chimneys shall enter from the side. Inlets shall have a thimble of fireclay, rigid refractory material or metal that will prevent the connector from pulling out of the inlet or from extending beyond the wall of the liner.

F.18—Masonry chimney cleanout openings

Cleanout openings shall be provided within 150 mm of the base of each flue within every masonry chimney. The upper edge of the cleanout shall be located not less than 150 mm below the lowest chimney inlet opening. The height of the opening shall be not less than 150 mm. The cleanout shall be provided with a noncombustible cover.

Exception: Chimney flues serving masonry fireplaces, where cleaning is possible through the fireplace opening.

F.19—Chimney clearances

Any portion of a masonry chimney located in the interior of the building or within the exterior wall of the building shall have a minimum airspace clearance to combustibles of 50 mm. Chimneys located entirely outside the exterior walls of the building, including chimneys that pass through the soffit or cornice, shall have a minimum airspace clearance of 25 mm. The airspace shall not be filled, except to provide fire blocking in accordance with Section F20.

Exceptions:

(1) Masonry chimneys equipped with a chimney lining system listed and labeled for use in chimneys in contact with combustibles in accordance with UL 1777, and installed in accordance with the manufacturer's instructions, are permitted to have combustible material in contact with their exterior surfaces.

(2) Where masonry chimneys are constructed as part of masonry or concrete walls, combustible materials shall not be in contact with the masonry or concrete wall less than 300 mm from the inside surface of the nearest flue lining.

(3) Exposed combustible trim and the edges of sheathing materials, such as wood siding, are permitted to abut the masonry chimney sidewalls, in accordance with Figure F.2, provided such combustible trim or sheathing is not less than 300 mm from the inside surface of the nearest flue lining. Combustible material and trim shall not overlap the corners of the chimney by more than 25 mm.

F.20—Chimney fire blocking

All spaces between chimneys and floors and ceilings through which chimneys pass shall be fire blocked with noncombustible material securely fastened in place. The fire blocking of spaces between wood joists, beams or headers shall be self-supporting or be placed on strips of metal or metal lath laid across the spaces between combustible material and the chimney.

TABLES AND FIGURES OF APPENDIX F

Table F.1: Net cross-sectional area of round flue sizes[a]

FLUE SIZE, INSIDE DIAMETER (mm)	CROSS-SECTIONAL AREA (mm^2)
150	17670
175	24050
200	31415
250	49090
269	56830
300	70685
375	110445
450	159045

[a] Flue sizes are based on ASTM C 315.

Table F.2: Net cross-sectional area of square and rectangular flue sizes

FLUE SIZE, OUTSIDE NOMINAL DIMENSIONS (mm)	CROSS-SECTIONAL AREA (mm^2)
114 × 216	14838
114 × 330	21935
203 × 203	27096
216 × 216	31613
203 × 305	43226
216 × 330	49032
305 × 305	65806
216 × 457	65161
330 × 330	81991
305 × 406	84516
330 × 457	111613
406 × 406	116774
406 × 508	143225
457 × 457	150322
508 × 508	192258
508 × 609	216129
609 × 609	278063

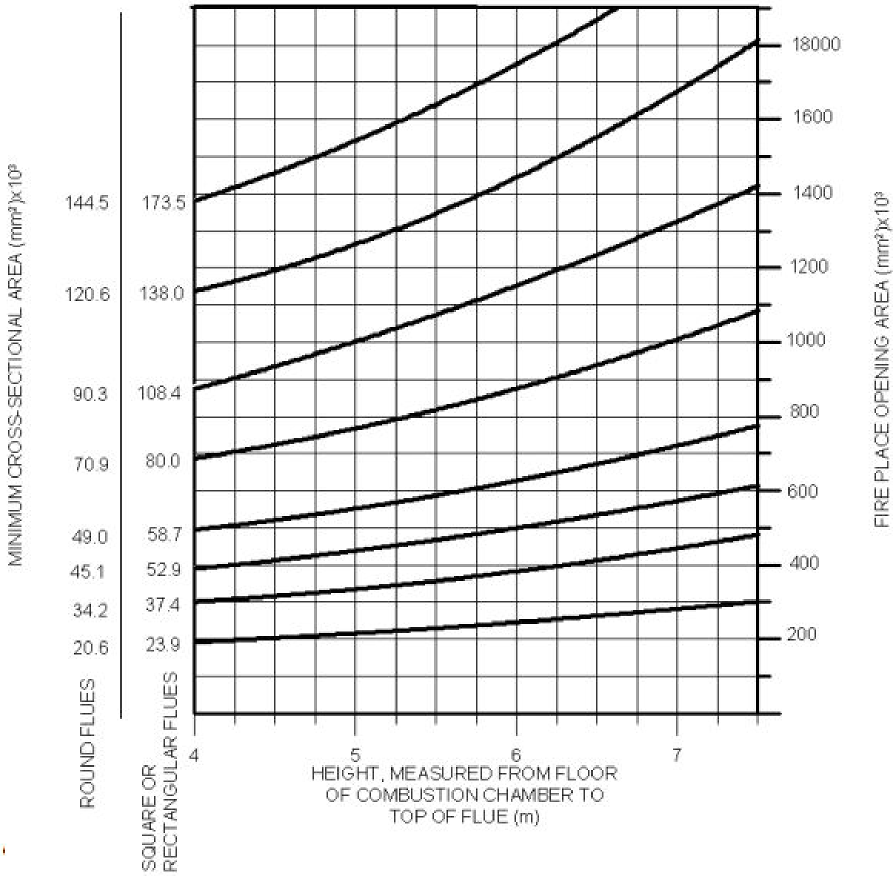


Figure F.1: Flue sizes for masonry chimneys

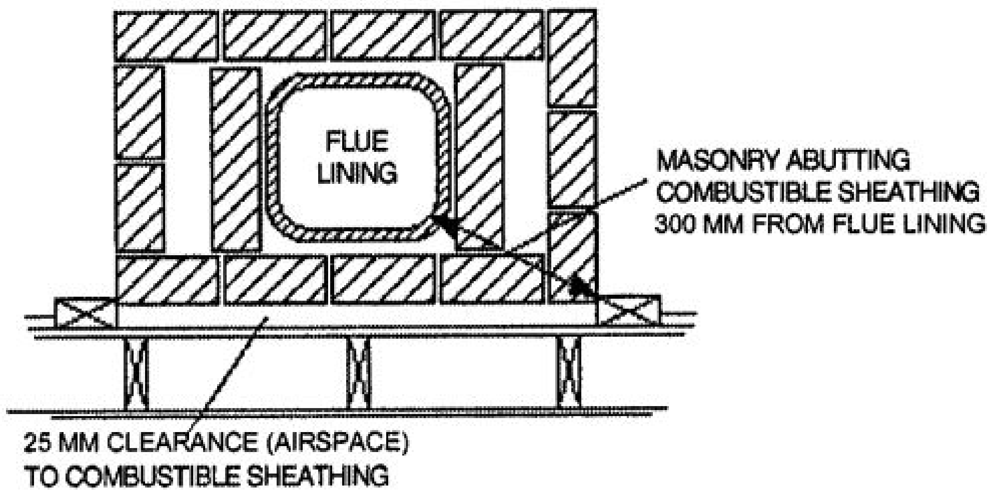


Figure F.2: Illustration of exception three chimney clearance provision