Building Code Requirements for Masonry Structures (TMS 402-xx)

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Building Code Requirements for Masonry Structures (TMS 402-~~16~~xx)

SYNOPSIS

This Code addresses the design and construction of masonry structures. It is written in such form that it may be adopted by reference in a legally adopted building code.

Among the subjects addressed are: definitions; 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; autoclaved aerated concrete masonry; masonry infill, veneers, glass unit masonry; and masonry partition walls. The Code Appendices address limit design and glass fiber reinforced polymer (GFRP) reinforced masonry.

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The quality, inspection, testing, and placement of materials used in construction are covered by reference to TMS 602-~~16~~xx Specification for Masonry Structures and other standards.

Keywords: AAC, masonry, allowable stress design, anchors (fasteners); anchorage (structural); autoclaved aerated concrete masonry, beams; building codes; cements; clay brick; clay tile; columns; compressive strength; concrete block; concrete brick; construction; detailing; flexural strength; glass fiber reinforced polym~~p~~er (GFRP); glass units; grout; grouting; infills; joints; loads (forces); limit design; masonry; masonry cements; masonry load bearing walls; masonry mortars; masonry walls; modulus of elasticity; mortars; pilasters; prestressed masonry, quality assurance; reinforced masonry; reinforcing steel; seismic requirements; shear strength; specifications; splicing; stresses; strength design, structural analysis; structural design; ties; unreinforced masonry; veneers; walls.

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PART 1: GENERAL

CHAPTER 1
GENERAL REQUIREMENTS

TMS 402 CODE

1.1 — Scope

1.1.1 *Minimum requirements*
This Code provides minimum requirements for the structural design and construction of masonry consisting of masonry units bedded in mortar.

1.1.2 *Governing building code*
This Code supplements the legally adopted building code and shall govern in matters pertaining to structural design and construction of masonry. In areas without a legally adopted building code, this Code defines the minimum acceptable standards of design and construction practice.

1.1.3 *SI information*
Numeric values are stated in inch-pound units and are to be regarded as standard. Values in parentheses are mathematical conversions to SI units provided for information only and are not to be considered as standard.

COMMENTARY

1.1 — Scope

1.1.1 *Minimum requirements*
This Code governs structural design of both structural and non-structural masonry. Examples of non-structural masonry are masonry veneer, glass unit masonry, and masonry partitions. Structural design aspects of non-structural masonry include, but are not limited to, gravity and lateral support, and load transfer to supporting members.

Masonry structures may be required to have enhanced structural integrity as part of a comprehensive design against progressive collapse due to accident, misuse, sabotage or other causes. General design guidance addressing this issue is available in Commentary Section 1.4 of ASCE/SEI 7. Suggestions from that Commentary, of specific application to many masonry structures, include but are not limited to: consideration of plan layout to incorporate returns on walls, both interior and exterior; use of load-bearing interior walls; adequate continuity of walls, ties, and joint rigidity; providing walls capable of beam action; ductile detailing and the use of compartmentalized construction.

Prior to the 2022 edition of this standard, provisions were included for empirically designed masonry. Given the advancement of the engineered design options for masonry coupled with the increasingly infrequent use of the empirical design provisions, the Committee has removed the empirical design option for designing new masonry structures.

1.1.3 *SI information*
The equivalent equations for use with SI units are provided in the Equation Conversions table in Part 5.

TMS 402 CODE

1.2 — Contract documents and calculations

1.2.1 Show or indicate all information required by TMS 402 on the project drawings or in the project specifications, 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.

(d) Size and location of structural members.

(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) Reinforcement 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.

(j) Masonry members in which mortar cement mortar or non-air-entrained cement-lime mortar is required.

COMMENTARY

1.2 — Contract documents and calculations

The provisions for preparation of project drawings, project specifications, and issuance of permits are, in general, consistent with those of most legally adopted building codes and are intended as supplements to those codes.

This Code is not intended to be made a part of the contract documents. The contractor should not be required through contract documents to assume responsibility for design (Code) requirements, unless the construction entity is acting in a design-build capacity.

1.2.1 This Code lists some of the more important items of information that must be included in the project drawings or project specifications. This is not an all-inclusive list, and additional items may be required by the building official.

(h) Control joints, expansion joints, and other movement joints are the primary means of accommodating dimensional changes and differential movement. Movement joint locations are recommended to be included on the project drawings as they may provide greater clarity than notes. Joint placement can influence structural design and performance in many ways, including, but not limited to, shear wall length, flange behavior at corners and/or intersecting walls, and potential interference with lintel bearing. Therefore, it is recommended that the drawings accurately reflect design assumptions so that the masonry and movement joints can be constructed and placed as intended. ~~Graphic depictions of movement joints may provide greater clarity than notes.~~ ~~Graphic depictions of joints may provide greater clarity compared to notes.~~

(j) Under certain conditions, masonry cement mortar is not permitted and either cement-lime mortar or mortar cement mortar must be used. Those conditions include, but are not limited to: masonry in which joint reinforcement is used to resist applied vertical and lateral loads (Sections 6.1.6.2.1.2 and 6.1.6.2.2.2) and participating masonry elements that are not fully grouted (Section 7.4.4.2.2).

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TMS 402 CODE

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.

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 — Alternative design or method of construction

The provisions of this Code and TMS 602 referenced within this Code are not intended to prohibit a design or exclude a method of construction within the scope of this Code and TMS 602 but not specifically prescribed therein, provided such design or method of construction has been approved by the authority having jurisdiction.

COMMENTARY

Refer to ASTM C270 for more information on mortar kinds and types.

1.2.2 Masonry design performed in accordance with engineered methods is based on the specified compressive strength of the masonry. For engineered masonry, structural adequacy of masonry construction requires that the compressive strength of masonry equals or exceeds the specified strength. Masonry compressive strength need not be verified when masonry is designed~~design~~ by prescriptive methods.~~approaches relies on rules and masonry compressive strength need not be verified~~.

1.2.3 The contract documents must accurately reflect design requirements. For example, joint and opening locations assumed in the design should be coordinated with locations shown on the drawings.

1.2.4 Verification that masonry construction conforms to the contract documents is required by this Code. A program of quality assurance must be included in the contract documents to satisfy this Code requirement.

1.3 — Alternative design or method of construction

New methods of design, new materials, and new uses of materials must undergo a period of development before being specifically addressed by a code. Hence, valid systems or components might be excluded from use by implication if means were not available to obtain acceptance. This section permits proponents to submit data substantiating the adequacy of their system or component to the authority having jurisdiction.

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TMS 402 CODE

1.4 — Standards cited in this Code

Standards of the American National Standards 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 602-22 — Specification for Masonry Structures

ANSI A137.1-19 — American National Standard Specifications for Ceramic Tile

ASCE/SEI 7-~~16~~22 — Minimum Design Loads and Associated Criteria for Buildings and Other Structures

ASTM A416/A416M-18 — Standard Specification for Low-Relaxation, Seven-Wire Steel Strand for Prestressed Concrete

ASTM A421/A421M-~~15~~21 — Standard Specification for Stress-Relieved Steel Wire for Prestressed Concrete

ASTM A706/A706M-16 — Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement

ASTM A722/A722M-18 — Standard Specification for High-Strength Steel Bars for Prestressed Concrete

ASTM C34-17 — Standard Specification for Structural Clay Loadbearing Wall Tile

ASTM C55-17 — Standard Specification for Concrete Building Brick

ASTM C56-13(2017) — Standard Specification for Structural Clay Nonloadbearing Tile

ASTM C62-17 — Standard Specification for Building Brick (Solid Masonry Units Made from Clay or Shale)

ASTM C73-17 — Standard Specification for Calcium Silicate Brick (Sand-Lime Brick)

ASTM C90-~~16a~~21 — Standard Specification for Loadbearing Concrete Masonry Units

ASTM C126-19 — Standard Specification for Ceramic Glazed Structural Clay Facing Tile, Facing Brick, and Solid Masonry Units

ASTM C140/140M-~~20a~~21 — Standard Test Methods for Sampling and Testing Concrete Masonry Units and Related Units

ASTM C212-~~20~~21 — Standard Specification for Structural Clay Facing Tile

ASTM C216-~~19~~21 — Standard Specification for Facing Brick (Solid Masonry Units Made from Clay or Shale)

ASTM C426-16 — Standard Test Method for Linear Drying Shrinkage of Concrete Masonry Units

COMMENTARY

1.4 — Standards cited in this Code

These standards are referenced in this Code. Specific dates are listed here because changes to the standard may result in changes of properties or procedures.

Contact information for these organizations is given below:

American Society of Civil Engineers (ASCE)
1801 Alexander Bell Drive
Reston, VA 20191
www.asce.org

American National Standards Institute (ANSI)
25 West 43rd Street, 4th Floor
New York, NY 10036
www.ansi.org

ASTM International
100 Barr Harbor Drive
West Conshohocken, PA 19428-2959
www.astm.org

American Welding Society (AWS)
8669 NW 36th Street, Suite 130
Miami, Florida 33166-6672
www.aws.org

The Masonry Society (TMS)
105 South Sunset Street, Suite Q
Longmont, Colorado 80501-6172
www.masonrysociety.org

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ASTM C476-20 — Standard Specification for Grout for Masonry

ASTM C482-20 — Standard Test Method for Bond Strength of Ceramic Tile to Portland Cement Paste

ASTM C652-~~19b~~21 — Standard Specification for Hollow Brick (Hollow Masonry Units Made from Clay or Shale)

ASTM C744-16 — Standard Specification for Prefaced Concrete and Calcium Silicate Masonry Units

ASTM C1006/1006M-20a — Standard Test Method for Splitting Tensile Strength of Masonry Units

ASTM C1088-20 — Standard Specification for Thin Veneer Brick Units Made from Clay or Shale

ASTM C1325-21 — Standard Specification for Fiber-Mat Reinforced Cementitious Backer Units

ASTM C1364-19 Standard Specification for Architectural Cast Stone

ASTM C1405-20a — Standard Specification for Glazed Brick (Single Fired, Brick Units)

ASTM C1611/C1611M-~~18~~21 — Standard Test Method for Slump Flow of Self-Consolidating Concrete

ASTM C1634-~~17~~20 — Standard Specification for Concrete Facing Brick and Other Concrete Masonry Facing Units

ASTM C1670/C1670M-~~20a~~21a — Standard Specification for Adhered Manufactured Stone Masonry Veneer Units

ASTM C1693-11 (2017) — Standard Specification for Autoclaved Aerated Concrete (AAC)

ASTM C1877-19 — Standard Specification for Adhered Concrete Masonry Units

ASTM C1892/~~C189220~~C1892M-20a — Standard Test Methods for Strength of Anchors in Masonry

ASTM E111-17 — Standard Test Method for Young's Modulus, Tangent Modulus, and Chord Modulus

AWS D 1.4/D1.4M: 2018 — Structural Welding Code — ~~Reinforcing~~ Steel Reinforcing Bars

COMMENTARY

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CHAPTER 2
NOTATION AND DEFINITIONS

TMS 402 CODE

2.1 — Notation

A_b = effective cross-sectional area of an anchor bolt, in.^{2} (mm^2)

A_{br} = bearing area, in.^{2} (mm^2)

A_f = area of GFRP reinforcement, in.^{2} (mm^2)

A_g = gross cross-sectional area of a member, in.^{2} (mm^2)

A_n = net cross-sectional area of a member, in.^{2} (mm^2)

A_{nv} = net shear area, in.^{2} (mm^2)

A_{ps} = area of prestressing steel, in.^{2} (mm^2)

A_{pt} = projected tension area on masonry surface of a right circular cone, in.^{2} (mm^2)

A_{pv} = projected shear area on masonry surface of one-half of a right circular cone, in.^{2} (mm^2)

A_s = area of nonprestressed longitudinal tension reinforcement, in.^{2} (mm^2)

A_{sc} = area of reinforcement placed within the lap, near each end of the lapped reinforcing bars or deformed wires and transverse to them, in.^{2} (mm^2)

A_{sp} = cross-sectional area of reinforcement within the net shear area, perpendicular to and crossing the horizontal shear plane, in.^{2} (mm^2)

A_{st} = total area of laterally tied longitudinal reinforcing steel, in.^{2} (mm^2)

A_t = tributary area of a veneer tie, ft^2 (m^2)

A_v = cross-sectional area of shear reinforcement, in.^{2} (mm^2)

A_1 = loaded area, in.^{2} (mm^2)

A_2 = supporting bearing area, in.^{2} (mm^2)

a = depth of an equivalent compression stress block at nominal strength, in. (mm)

B_a = allowable axial load on an anchor bolt, lb (N)

B_{ab} = allowable axial tensile load on an anchor bolt when governed by masonry breakout, lb (N)

B_{an} = nominal axial strength of an anchor bolt, lb (N)

B_{anb} = nominal axial tensile strength of an anchor bolt when governed by masonry breakout, lb (N)

B_{anp} = nominal axial tensile strength of an anchor bolt when governed by anchor pullout, lb (N)

COMMENTARY

2.1 — Notation

Notations used in this Code are summarized here.

TMS 402 CODE

B_{ans} = nominal axial tensile strength of an anchor bolt when governed by the tensile strength of the steel, lb (N)

B_{ap} = allowable axial tensile load on an anchor bolt when governed by anchor pullout, lb (N)

B_{as} = allowable axial tensile load on an anchor bolt when governed by the tensile strength of the steel, lb (N)

B_v = allowable shear load on an anchor bolt, lb (N)

B_{vb} = allowable shear load on an anchor bolt when governed by masonry breakout, lb (N)

B_{vc} = allowable shear load on an anchor bolt when governed by masonry crushing, lb (N)

B_{vn} = nominal shear strength of an anchor bolt, lb (N)

B_{vnb} = nominal shear strength of an anchor bolt when governed by masonry breakout, lb (N)

B_{vnc} = nominal shear strength of an anchor bolt when governed by masonry crushing, lb (N)

B_{vnpry} = nominal shear strength of an anchor bolt when governed by anchor pryout, lb (N)

B_{vns} = nominal shear strength of an anchor bolt when governed by the shear strength of the steel, lb (N)

B_{vpry} = allowable shear load on an anchor bolt when governed by anchor pryout, lb (N)

B_{vs} = allowable shear load on an anchor bolt when governed by the shear strength of the steel, lb (N)

b = width of section, in. (mm)

b_a = total applied design axial force on an anchor bolt, lb (N)

b_{au} = strength level axial load on an anchor bolt, lb (N)

b_v = total applied design shear force on an anchor bolt, lb (N)

b_{vu} = strength level shear load on an anchor bolt, lb (N)

b_w = width of wall beam, in. (mm)

b_{web} = width of unit web, in. (mm)

C = lesser of the cover to the center of the bar or one-half of the center-to-center spacing of the bars being developed, in. (mm)

~~C_d = deflection amplification factor~~

C_E = environmental reduction factor

c = distance from the fiber of maximum compressive strain to the neutral axis, in. (mm)

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c_b = depth to neutral axis at balanced conditions, in. (mm)

D = dead load or related internal moments and forces

d = distance from extreme compression fiber to centroid of nonprestressed tension reinforcement, in. (mm)

d_b = nominal diameter of reinforcement or anchor bolt, in. (mm)

d_o = nominal anchor diameter, in. (mm)

d_{ps} = distance from extreme compression fiber to centroid of prestressed reinforcement, in. (mm)

d_v = actual depth of a member in direction of shear considered, in. (mm)

E = load effects of earthquake or related internal moments and forces

E_{AAC} = modulus of elasticity of AAC masonry in compression, psi (MPa)

E_{bb} = modulus of elasticity of bounding beams, psi (MPa)

E_{bc} = modulus of elasticity of bounding columns, psi (MPa)

E_f = modulus of elasticity of GFRP reinforcement, psi (MPa)

E_h = affect of horizontal seismic forces

E_m = modulus of elasticity of masonry in compression, psi (MPa)

E_{mCS} = modulus of elasticity of cast stone masonry in compression, psi (MPa)

E_{ps} = modulus of elasticity of prestressing steel, psi (MPa)

E_s = modulus of elasticity of steel, psi (MPa)

E_v = affect of vertical seismic forces

e = eccentricity of axial load, in. (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, in. (mm)

e_u = eccentricity of P_{uf}, in. (mm)

F_a = allowable compressive stress available to resist axial load only, psi (MPa)

F_b = allowable compressive stress available to resist flexure only, psi (MPa)

F_f = allowable shear-friction stress, psi (MPa)

COMMENTARY

TMS 402 CODE

COMMENTARY

F_s = allowable tensile or compressive stress in reinforcement, psi (MPa)

F_v = allowable shear stress, psi (MPa)

F_{vm} = allowable shear stress resisted by the masonry, psi (MPa)

F_{vs} = allowable shear stress resisted by the shear reinforcement, psi (MPa)

f_a = calculated compressive stress in masonry due to axial load only, psi (MPa)

f_b = calculated compressive stress in masonry due to flexure only, psi (MPa)

f_f = stress in GFRP reinforcement at masonry crushing limit state, psi (MPa)

f_{fb} = design tensile strength of bent portion of GFRP bar, psi (MPa)

f_{fd} = design tensile strength for GFRP reinforcement, psi (MPa)

f_{fr} = required GFRP reinforcing bar stress, psi (MPa)

f_{fu} = tensile strength for product certification as reported by GFRP manufacturer, psi (MPa)

f_h = specified unit compressive strength of the clay masonry or concrete masonry header based on net cross-sectional area, psi (MPa)

f'_{AAC} = specified compressive strength of AAC masonry, psi (MPa)

f'_g = specified compressive strength of grout, psi (MPa)

f'_m = specified compressive strength of clay masonry or concrete masonry, psi (MPa)

f'_{mi} = specified compressive strength of clay masonry or concrete masonry at the time of prestress transfer, psi (MPa)

f_{ps} = stress in prestressing tendon at nominal strength, psi (MPa)

f_{pu} = specified tensile strength of prestressing tendon, psi (MPa)

f_{py} = specified yield strength of prestressing tendon, psi (MPa)

f_r = modulus of rupture, psi (MPa)

f_{rAAC} = modulus of rupture of AAC, psi (MPa)

f_s = calculated tensile or compressive stress in reinforcement, psi (MPa)

f_{se} = effective stress in prestressing tendon after prestress losses, psi (MPa)

TMS 402 CODE

$f_{t\,AAC}$ = splitting tensile strength of AAC as determined in accordance with ASTM C1006, psi (MPa)

f_u = specified minimum tensile strength, psi (MPa)

f_v = calculated shear stress in masonry, psi (MPa)

f_{vd} = direct shear strength, psi (MPa)

f_y = specified yield strength of steel for reinforcement and anchors, psi (MPa)

G = modulus of rigidity (shear modulus) of masonry, psi (MPa)

h = effective height of column, wall, or pilaster, in. (mm)

h_b = unsupported height of backing; for cantilevers, the unsupported height shall be taken as twice the height of the cantilever, ft (m)

h_{inf} = vertical dimension of infill, in. (mm)

h_w = height of entire wall or of the segment of wall considered, in. (mm)

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

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

I_{cr} = moment of inertia of cracked cross-sectional area of a member, in.^{4} (mm^4)

I_{eff} = effective moment of inertia, in.^{4} (mm^4)

I_g = moment of inertia of gross cross-sectional area of a member, in.^{4} (mm^4)

I_n = moment of inertia of net cross-sectional area of a member, in.^{4} (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, in. (mm)

K_{AAC} = dimension used to calculate reinforcement development for AAC masonry, in. (mm)

k = ratio of the distance between the neutral axis and the extreme fiber in compression to the depth, d, using the assumptions of Section 8.3.2

k_b = GFRP reinforcement bar bend coefficient

k_c = coefficient of creep of masonry, per psi (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 Fahrenheit (degree Celsius)

COMMENTARY

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TMS 402 CODE

k_{tie} = stiffness of veneer tie, lb/ft (N/mm)

L = live load or related internal moments and forces

ℓ = clear span between supports, in. (mm)

ℓ_b = effective embedment length of headed or bent bar anchor bolts, in. (mm)

ℓ_{be} = anchor bolt edge distance, in. (mm)

ℓ_d = development length or lap length of straight reinforcement, in. (mm)

ℓ_{dh} = development length of hooked reinforcement measured from the outside of the bar or wire at the hook, in. (mm)

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

ℓ_{eff} = effective span length for a deep beam, in. (mm)

ℓ_{inf} = plan length of infill, in. (mm)

ℓ_p = clear span of the prestressed member in the direction of the prestressing tendon, in. (mm)

ℓ_w = length of entire wall or of the segment of wall considered in direction of shear force, in. (mm)

M = maximum moment at the section under consideration, in.-lb (N-mm)

M_a = maximum moment in member due to the applied allowable stress level loading for which deflection is calculated, in.-lb (N-mm)

M_{cr} = nominal cracking moment strength, in.-lb (N-mm)

M_n = nominal moment strength, in.-lb (N-mm)

M_s = moment under allowable stress level loads, including P-delta effects, at midheight of a member, in.-lb (N-mm)

M_u = strength level moment, magnified by second-order effects where required by this Code, in.-lb (N-mm)

$M_{u,0}$ = strength level moment from first-order analysis, in.-lb (N-mm)

n = modular ratio, E_s/E_m

n_t = number of threads per inch

P = axial load, lb (N)

P_a = allowable axial compressive force in a reinforced member, lb (N)

P_{bal} = nominal axial strength at balanced conditions

P_c = allowable total horizontal projection of corbelling, in. (mm)

COMMENTARY

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TMS 402 CODE

P_e = Euler buckling load, lb (N)

P_n = nominal axial strength, lb (N)

P_{ps} = prestressing tendon force at time and location relevant for design, lb (N)

P_u = strength level axial load, lb (N)

P_{uf} = strength level load from tributary floor or roof areas, lb (N)

P_{uw} = strength level load from weight of wall area tributary to wall section under consideration, lb (N)

p_{allow} = allowable stress level out-of-plane load, psf (Pa)

p_u = strength level out-of-plane load, psf (Pa)

p_{unit} = allowable projection of one unit, in. (mm)

p_{veneer} = strength level design wind pressure on veneer as determined from ASCE/SEI 7, Chapter 30, psf (kPa)

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, in.^{3} (mm^3)

Q_E = the effect of horizontal seismic forces from the base shear

$q_{n\ inf}$ = nominal out-of-plane flexural capacity of infill per unit area, psf (Pa)

R = response modification coefficient

r = radius of gyration, in. (mm)

S = snow load or related internal moments and forces

S_n = section modulus of the net cross-sectional area of a member, in.^{3} (mm^3)

s = spacing of reinforcement, in. (mm)

s_l = total linear drying shrinkage of concrete masonry units determined in accordance with ASTM C426

t = nominal thickness of member, in. (mm)

t_{fs} = minimum specified thickness of face shell, in. (mm)

t_{inf} = specified thickness of infill, in. (mm)

$t_{net\ inf}$ = net thickness of infill, in. (mm)

t_{sp} = specified thickness of member, in. (mm)

V = shear force, lb (N)

V_{cr} = flexural cracking strength, lb (N)

V_{lim} = limiting base-shear strength, lb (N)

V_n = nominal shear strength, lb (N)

V_{nAAC} = nominal shear strength provided by AAC masonry,

COMMENTARY

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lb (N)

V_{nf} = nominal shear-friction strength, lb (N)

COMMENTARY

TMS 402 CODE

$V_{n\,inf}$ = nominal horizontal in-plane shear strength of infill, lb (N)

V_{nm} = nominal shear strength provided by masonry, lb (N)

V_{ns} = nominal shear strength provided by shear reinforcement, lb (N)

V_u = strength level shear load, lb (N)

V_{ub} = base-shear demand, lb (N)

v = shear stress, psi (MPa)

W = wind load or related internal moments and forces

w_{inf} = width of equivalent strut, in. (mm)

w_{strut} = horizontal projection of the width of the diagonal strut, in. (mm)

w_u = out-of-plane strength level uniformly distributed load, lb/in. (N/mm)

x_t = centroid of the tension force from the compression face, in. (mm)

z = internal lever arm between compressive and tensile forces in a deep beam, in. (mm)

α_{arch} = horizontal arching parameter for infill, $lb^{0.25}$ ($N^{0.25}$)

β_{arch} = vertical arching parameter for infill, $lb^{0.25}$ ($N^{0.25}$)

β_b = ratio of area of reinforcement cut off to total area of tension reinforcement at a section

γ = reinforcement size factor for straight development length

γ_d = parameter to account for variation in stiffness along the length of the member

γ_g = grouted shear wall factor

γ_h = reinforcement size factor for hooked development length

Δ = ~~calculated~~design story drift, in. (mm)

Δ_a = allowable story drift, in. (mm)

~~δ_{ne} = displacements calculated using code-prescribed seismic forces and assuming elastic behavior, in. (mm)~~

δ_{MCE} = displacement due to Maximum Considered Earthquake as defined in ASCE/SEI 7, in. (mm)

δ_s = horizontal deflection at midheight under allowable stress level loads, in. (mm)

δ_{ser} = deflection due to service level loads, in. (mm)

δ_u = deflection due to strength level loads, in. (mm)

ε_{cs} = drying shrinkage of AAC

ε_{mu} = maximum usable compressive strain of masonry

COMMENTARY

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ε_t = net tensile strain in extreme layer of longitudinal tension reinforcement at nominal strength

ε_{ty} = value of net tensile strain in extreme layer of longitudinal tension reinforcement used to define a compression-controlled section

ε_{fd} = design tensile strain for GFRP reinforcement

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ε_{ft} = tensile strain in GFRP reinforcement at section limit state

ε_{fu} = tensile strain at rupture for product certification as reported by GFRP manufacturer

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, in.$^{-1}$ (mm^{-1})

μ = coefficient of friction

μ_{AAC} = coefficient of friction of AAC

~~γ_d = parameter to account for variation in stiffness along the length of the member~~

ρ = reinforcement ratio

ρ_f = reinforcement ratio of GFRP reinforcement

ρ_{max} = maximum flexural tension reinforcement ratio

ϕ = strength-reduction factor

$\cancel{X}\chi$ = factor used to account for direction of compressive stress in a masonry member relative to the direction used for the determination of f'_m

ψ = magnification factor for second-order effects

COMMENTARY

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TMS 402 CODE

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 specified 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 specified dimensions.

Area, net shear — The net area that is effective in resisting shear.

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 — Structural wall or surface to which veneer is attached. ~~Backings include concrete, masonry, and light frame. Light frame backings consist of either wood studs or cold-formed metal studs with associated auxiliary members.~~

.

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

Beam — A member designed primarily to resist flexure and shear induced by loads perpendicular to its longitudinal axis.

COMMENTARY

2.2 — Definitions

For consistent application of this Code, terms are defined that have particular meanings in this Code. The definitions given are for use in application of this Code only and do not always correspond to ordinary usage. Other terms are defined in referenced documents and those definitions are applicable. If any term is defined in both this Code and in a referenced document, the definition in this Code applies. Referenced documents are listed in Section 1.4. Terminology standards include ASTM C1232 Standard Terminology for Masonry and ASTM C1180 Standard Terminology of Mortar and Grout for Unit Masonry. Glossaries of masonry terminology are available from several sources within the industry (BIA TN 2 (1999); NCMA TEK 1-4 (2004); and IMI (1981)).

Area, net shear — Refer to Table ~~4.3~~4.4.5 for a graphical explanation of the net shear area for fully and partially grouted reinforced masonry members.

Backing — The structural role provided by the backing varies between anchored and adhered veneer systems. For anchored veneer, the backing provides lateral support. For adhered veneer, the backing provides lateral and vertical support.

Backings typically are concrete, masonry, and light frame. In the context of this ~~code~~Code, the use of the term “light frame backing” refers to wood or cold-formed metal studs and other structural members, such as rim joists, used in light frame construction.

Beam — A beam usually spans horizontally, although it may have another orientation in space. For the gravity load resisting system, beams primarily resist flexural and shear loads. However, a beam may be required to resist axial loads.

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TMS 402 CODE

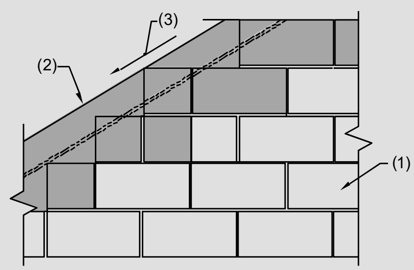
Bond beam — A horizontal, sloped, or stepped member that is fully grouted, has longitudinal reinforcement, and is constructed within a masonry wall.

COMMENTARY

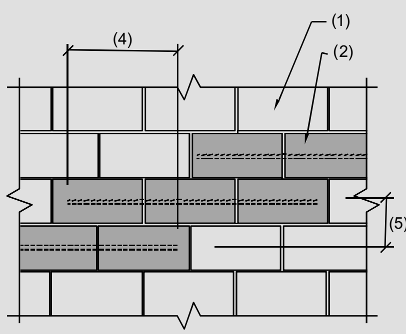
Bond beam – This reinforced member is usually constructed horizontally, but may be sloped or stepped to match an adjacent roof, for example, as shown in Figure CC-2.2-1.

Notes:

(1) Masonry wall
(2) Fully grouted bond beam with reinforcement
(3) Sloped top of wall
(4) Length of noncontact lap splice
(5) Spacing between bars or deformed wire in noncontact lap splice



(a) Sloped Bond Beam
(not to scale)



(b) Stepped Bond Beam
(not to scale)

Figure CC-2.2-1 — Sloped and stepped bond beams

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Bonded prestressing tendon — Prestressing tendon encapsulated by prestressing grout in a corrugated duct that is bonded to the surrounding masonry through grouting.

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 a duly authorized representative.

Cavity — The space between wythes of non-composite masonry or between a masonry veneer and its backing~~, which may contain insulation~~.

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

Cement backer unit — A rigid panel made of portland cement, aggregate, and glass mesh complying with ASTM C1325, Type A or B as applicable.

Collar joint — Vertical longitudinal space between wythes of composite masonry that is 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 connected 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.

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

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

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

COMMENTARY

Cavity — A cavity can be part of a multiwythe masonry wall assuming non-composite action (Section 5.1.~~4~~3.3) or a veneer wall (Chapter 13). The cavity ~~may be detailed as~~ includes a drainage space and may contain sheathing, insulation, drainage mats, fasteners, veneer ties, wall ties, mortar collection devices, and ancillary accessories depending upon function and design intent. Cavities are not permitted to be spanned by headers and are not permitted to be filled solid with mortar or grout, except for cavities below the base flashing which should be filled solid with mortar or grout. Also see "Drainage Space" to differentiate between the two definitions.

Cement backer unit — Material complying with ASTM C1325 Type A is intended for exterior applications and material complying with ASTM C1325 Type B is intended for interior applications.

Column — Generally, a column spans vertically, though it may have another orientation in space.

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Commented [PJS37]: 20-VG-039

Commented [PJS40]: 20-VG-039

Commented [PJS38]: 20-VG-039

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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.

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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 ASCE/SEI 7. ~~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 ASCE/SEI 7.~~

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Design strength — The nominal strength of a member 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. Thickness is given first, followed by height and then length.

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

Drainage space – A space within the cavity that allows for the drainage of water.

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.

Effective prestress — Stress remaining in prestressing tendons after all losses have occurred.

Fastener – A device that attaches a veneer tie, lath, or cement backer unit to the backing or that attaches a glass unit panel anchor to its support.

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.

COMMENTARY

Dimension, nominal — Nominal dimensions are usually used to identify the size of a masonry unit. The permitted tolerances for units are given in the appropriate material standards. Permitted tolerances for joints and masonry construction are given in TMS 602.

Dimensions, specified — Specified dimensions are most often used for design calculations.

Drainage space – The drainage space may contain materials such as mortar droppings, mortar protrusions, drainage media, veneer ties, and mortar dropping collection devices provided that moisture is able to drain from the space. See Figure CC-13.2-4.

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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.

Grout space, gross — The area or dimensions available within the continuous grouted cell, core, bond beam course, or collar joint, considering the effect of unit offset in adjacent courses but neglecting possible mortar protrusions and the presence of perpendicular reinforcement, if any.

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

COMMENTARY

Grout space, gross — The gross grout space is used to evaluate design limitations on reinforcement size and quantity. Depending upon the configuration of the masonry unit, the gross grout space in running bond may be smaller than the gross grout space in stack bond. Refer to Figure CC-2.2-2 for an example.

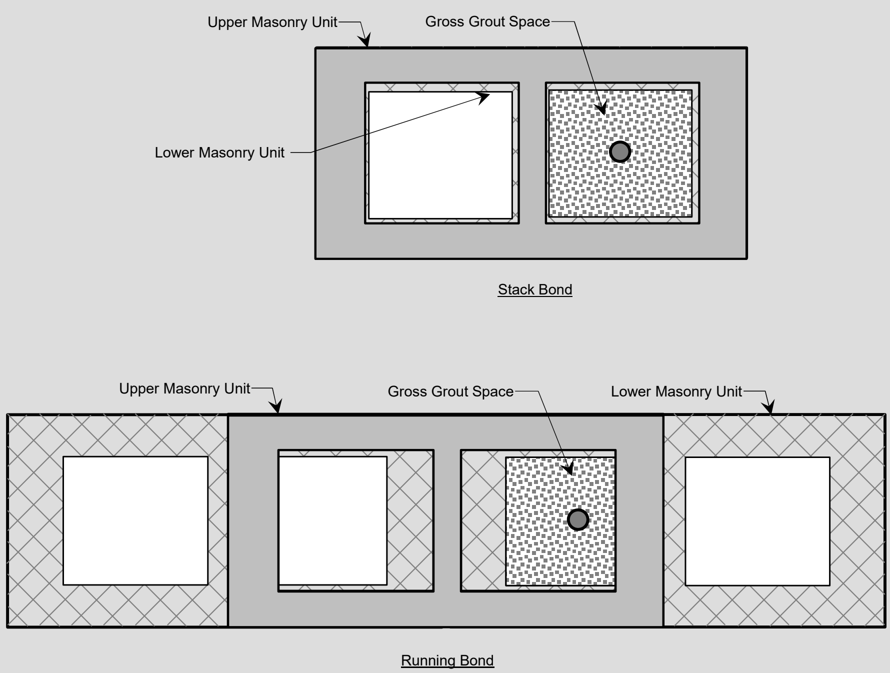


Figure CC-2.2-2 – Gross grout space in stack bond and in running bond

TMS 402 CODE

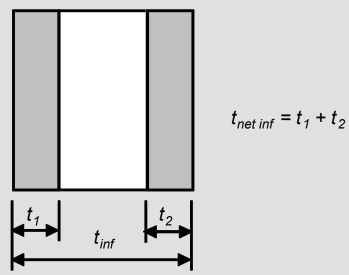
COMMENTARY

Header — 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, net thickness – The net thickness is shown in Figure CC-2.2-3



Vertical Section through Hollow Unit in Infill Wall

Figure CC-2.2-3 — Thickness and net thickness 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 — Special inspection by the special inspector who is present continuously when and where the work to be inspected is being performed.

Inspection, continuous — The Inspection Agency is required to be on the project site whenever masonry tasks requiring continuous inspection are in progress.

Inspection, periodic — Special inspection by the special inspector who is intermittently present where the work to be inspected has been or is being performed.

Inspection, periodic — During construction requiring periodic inspection, the Inspection Agency is only required to be on the project site intermittently, and is required to observe completed work. The frequency of periodic inspections should be defined by the Architect/Engineer as part of the quality assurance plan, and should be consistent with the complexity and size of the project.

Laterally restrained prestressing tendon — Prestressing tendon that is not free to move laterally within the cross section of the member.

Laterally unrestrained prestressing tendon — Prestressing tendon that is free to move laterally within the cross section of the member.

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*~~.

Licensed design professional — For convenience, the Commentary uses the term “designer” when referring to the licensed design professional - also referred to as a “registered design professional” in other codes and standards.

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Lintel — See Beam.

Lintel — The term “lintel” generally refers to a horizontal member over an opening, chase or recess. Masonry lintels are required to be designed in accordance with the beam provisions of this Code.

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Load, allowable stress level – Loads resulting from allowable stress design load combinations.

Load, dead — Dead weight supported by a member, as defined by the legally adopted building code.

Load, live — Live load specified by the legally adopted building code.

Load, strength level – Loads resulting from strength design load combinations.

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, plain — Shear walls designed using the unreinforced masonry design methods of this ~~code~~Code.

Masonry, reinforced — Masonry in which reinforcement acting in conjunction with the masonry is used in design to resist forces.

Masonry, unreinforced — Masonry in which reinforcement, if present, is not used in design to resist forces~~the tensile resistance of masonry is taken into consideration and the resistance of reinforcing steel, if present, is neglected~~.

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 the proportional limit of the material.

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

Mortar — (1) A plastic mixture of cementitious materials, fine aggregates, and water, with or without admixtures, that is used to construct unit masonry assemblies. (2) The hardened equivalent of such mixtures.

Nominal strength — The strength of a member 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.

COMMENTARY

Masonry, plain — This term is used to refer to two types of shear walls, as outlined in Chapter 7, and is used for consistency with ASCE/SEI 7 and IBC provisions.

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Pilaster — A vertical member, built integrally with a wall, with a portion of its cross section typically projecting from one or both faces of the wall.

Post-tensioning — Method of prestressing in which prestressing tendons are tensioned after the masonry has been placed.

Prestressed masonry — Masonry in which internal compressive stresses have been introduced by prestressed tendons to counteract potential tensile stresses in masonry resulting from applied loads.

Prestressing grout — A cementitious mixture used to encapsulate bonded prestressing tendons.

Prestressing tendon — Steel component such as wire, bar, strand, or a bundle of such components, used to impart prestress to masonry.

Pretensioning — Method of prestressing in which prestressing tendons are tensioned before the transfer of stress into the masonry.

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~~ graphical representations that, along with the project specifications, complete the descriptive information for constructing the work ~~required by the contract documents~~.

Commented [PJS49]: 19-GR-133

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.

Reinforcement, GFRP — Glass fiber reinforced polymer reinforcement

Reinforcement, shear — Reinforcement required for compliance with Section 8.3.5, Section 9.3.3.1.2, or Section 11.3.4.1.2.

Required strength — The strength needed to resist strength level loads.

COMMENTARY

Pilaster — A pilaster may support axial loads parallel to its longitudinal axis, as well as transverse loads applied perpendicular to its longitudinal axis. A projecting pilaster may or may not have longitudinal reinforcement, but non-projecting pilasters must be reinforced. Longitudinal reinforcement in a pilaster only needs to be laterally tied if the design relies upon that reinforcement to resist axial and/or flexural compression, although lateral ties (stirrups) may also be required if shear stresses are high. This Code allows for the effects of the "flanges" formed by the wall to be considered as part of the projecting pilaster, subject to certain requirements.

TMS 402 CODE

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

Scratch coat — the first layer of mortar applied over lath or other substrate in an adhered veneer.

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 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 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, intermediate reinforced prestressed masonry — A prestressed 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 AAC masonry — An AAC masonry shear wall designed to resist lateral forces while neglecting stresses in reinforcement, if present.

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

Shear wall, ordinary plain prestressed masonry — A prestressed 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.

COMMENTARY

Running bond — This Code concerns itself only with the structural effect of the masonry bond pattern. Therefore, the only distinction made by this Code is between masonry laid in running bond and masonry that is not laid in running bond. For purposes of this Code, architectural bond patterns that do not satisfy this Code definition of running bond are classified as not running bond. Masonry laid in other bond patterns must be reinforced to provide continuity across the head joints. Stack bond, which is commonly interpreted as a pattern with aligned heads joints, is one bond pattern that is required to be reinforced horizontally.

TMS 402 CODE

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.

Shear wall, special reinforced prestressed masonry — A prestressed 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 project specifications or project drawings, 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 psi (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 project specifications or project drawings, 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 psi (MPa).

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

Stone, cast (architectural cast stone) — an architectural precast concrete building unit manufactured to simulate dimension stone.

Stone, dimension — natural stone that has been selected and fabricated to specific sizes or shapes.

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

Tendon anchorage — In post-tensioning, a device used to anchor the prestressing tendon to the masonry or concrete member; in pretensioning, a device used to anchor the prestressing tendon during hardening of masonry mortar, grout, prestressing grout, or concrete.

Tendon coupler — A device for connecting two tendon ends, thereby transferring the prestressing force from end to end.

Tendon jacking force — Temporary force exerted by a device that introduces tension into prestressing tendons.

COMMENTARY

Special boundary elements — Requirements for longitudinal and transverse reinforcement have not been established in general and must be verified by testing. Research in this area is ongoing.

TMS 402 CODE

Thin-bed mortar — Mortar for use in construction of AAC unit masonry whose joints shall not be less than $^{1}/_{16}$ in. (1.5 mm).

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

Tie, veneer — Metal connector that attaches masonry veneer to backing.

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

Transfer — Act of applying to the masonry member the force in the prestressing tendons.

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

Unbonded prestressing tendon — Prestressing tendon that is not bonded to masonry.

Veneer, adhered — Masonry veneer secured to and supported by the backing through direct bond to a masonry or concrete backing; or bond to either a scratch coat and lath or a cement backer unit that is fastened to ~~a masonry, concrete, or light frame~~ the backing.

Veneer, anchored — Masonry veneer secured to and supported laterally by the backing through veneer ~~tiess~~ ties and supported vertically by the foundation or other structural members.

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 member, usually vertical , used to enclose or separate spaces or uses.

Wall, load-bearing — Wall supporting vertical loads greater than 200 lb/linear ft (2919 N/m) in addition to its own weight.

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.

COMMENTARY

TMS 402 Code and Commentary, C-31

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CHAPTER 3
QUALITY AND CONSTRUCTION

TMS 402 CODE

3.1 — Quality Assurance program

The quality assurance program shall comply with the Level defined in Table 3.1, depending on how the masonry was designed and the Risk Category, as defined in ASCE/SEI 7 or the legally adopted building code. 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, and shall comply with the minimum requirements of TMS 602, Tables 3 and 4, for the required Level.

Table 3.1: Minimum Quality Assurance Level

Designed in accordance with	Risk Category I, II or III	Risk Category IV
Part 3 or Appendix C or Appendix D	Level 2	Level 3
Part 4	Level 1	Level 2

COMMENTARY

3.1 — Quality Assurance program

Masonry design provisions in this Code are valid when the quality of masonry construction meets or exceeds that described in TMS 602. Therefore, in order to design masonry by this Code, verification of good quality construction is required. The means by which the quality of construction is monitored is the quality assurance program.

A quality assurance program must be defined in the contract documents, to answer questions such as “how to”, “what method”, “how often”, and “who determines acceptance”. This information is part of the administrative and procedural requirements. Typical requirements of a quality assurance program include review of material certifications, field inspection, and testing. The acts of providing submittals, inspecting, and testing are part of the quality assurance program.

Because the design and the complexity of masonry construction vary from project to project, so must the extent of the quality assurance program. The contract documents must indicate the testing, Special Inspection, and other measures that are required to assure that the Work is in conformance with the project requirements.

Section 3.1 establishes the minimum criteria required to assure that the quality of masonry construction conforms to the quality upon which the Code-permissible values are based. The scope of the quality assurance program depends on whether the structure is a Risk Category IV structure or not, as defined by ASCE/SEI 7 or the legally adopted building code. Because of their importance, Risk Category IV structures are subjected to more extensive quality assurance measures.

The level of required quality assurance depends on whether the masonry was designed in accordance with Part 3 or Appendix C (engineered), or in accordance with Part 4 (prescriptive).

TMS 602 Quality Assurance Table 4 requires inspection tasks to be performed on a continuous or periodic basis. The Architect/Engineer should define the required timing of periodic inspections so that they are sufficient to verify a representative sample of the materials and workmanship. The frequency of periodic inspection varies depending on the size and complexity of the project.

Level 1 Quality Assurance

For most projects, Level 1 only requires verification of compliance of submittals with the specified requirements. However, periodic inspection of veneers is required when the height of the veneer exceeds 60 ft. (18.3 m) above the grade plane.

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Level 2 Quality Assurance

Implementation of verification and inspection requirements contained in TMS 602 Tables 3 and 4 requires detailed knowledge of the appropriate procedures. Comprehensive verification and inspection procedures are available from recognized industry sources (Chrysler (~~2010~~2017); NCMA (2008); BIA (2001); BIA (1988)), which may be referenced for assistance in developing and implementing a Quality Assurance program. Certain applications, such as Masonry Veneer (Chapter 13), and Masonry Partition Walls (Chapter 15)~~)~~, do not require compressive strength verification of masonry.

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Installation techniques for AAC masonry and thin-bed mortar differ from concrete and clay masonry. Once it has been demonstrated in the field that compliance is attained for the installation of AAC masonry and thin-bed mortar, the frequency of Special Inspection may be revised from continuous to periodic. However, the frequency of Special Inspection should revert to continuous for the prescribed period whenever new AAC masonry installers work on the project.

Level 3 Quality Assurance

Premixed mortars and grouts are delivered to the project site as "trowel ready" or "pourable" materials, respectively. Preblended mortars and grouts are dry combined materials that are mixed with water at the project site. Verification of proportions of premixed or preblended mortars and grouts can be accomplished by review of manufacture's batch tickets (if applicable), a combination of preconstruction and construction testing, or other acceptable documentation.

3.1.1 *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.

3.1.1 *Procedures*

In addition to specifying testing and Special Inspection requirements, the quality assurance program must define the procedures for submitting the testing and inspection reports (that is, how many copies and to whom) and define the process by which those reports are to be reviewed.

Testing and evaluation should be addressed in the quality assurance program. The program should allow for the selection and approval of a testing agency, which agency should be provided with prequalification test information and the rights for sampling and testing of specific masonry construction materials in accordance with referenced standards. The evaluation of test results by the testing agency should indicate compliance or noncompliance with a referenced standard.

Further quality assurance evaluation should allow an appraisal of the testing program and the handling of nonconformance. Acceptable values for all test methods should be given in the contract documents.

Identification and resolution of noncomplying conditions should be addressed in the contract documents. A responsible person should be identified to allow resolution of nonconformances. In agreement with others in

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

3.1.3 *Acceptance relative to strength requirements*

3.1.3.1 *Compliance with specified compressive strength* — Compressive strength of masonry shall be considered satisfactory if the compressive strength of each masonry wythe and grouted collar joint equals or exceeds the applicable value of f'_m or f'_{AAC}.

3.1.3.2 *Determination of compressive strength* — Compressive strength of masonry shall be determined in accordance with the provisions of TMS 602.

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the design/construct team, the resolutions should be repaired, reworked, accepted as is, or rejected. Repaired and reworked conditions should initiate a reinspection.

Records control should be addressed in the contract documents. The distribution of documents during and after construction should be delineated. The review of documents should persist throughout the construction period so that each party is informed and that records for documenting construction occurrences are available and correct after construction has been completed.

3.1.2 *Qualifications*
The entities verifying compliance must be competent and knowledgeable of masonry construction and the requirements of this Code. Therefore, minimum qualifications for those individuals must also be established by the quality assurance program in the contract documents.

The responsible party performing the quality control measures should document the organizational representatives who will be a part of the quality control segment, their qualifications, and their precise conduct during the performance of the quality assurance phase.

ASTM C1093 defines the duties and responsibilities of testing agency personnel and defines the technical requirements for equipment used in testing masonry materials. Testing agencies who are accredited or inspected for conformance to the requirements of ASTM C1093 by a recognized evaluation authority are qualified to test masonry.

3.1.3 *Acceptance relative to strength requirements*
Fundamental to the structural adequacy of masonry construction is the necessity that the compressive strength of masonry equals or exceeds the specified strength. Rather than mandating design based on different values of f'_m or f'_{AAC} for each wythe of a multiwythe wall construction made of differing material, this Code requires the strength of each wythe and of grouted collar joints to equal or exceed f'_m or f'_{AAC} for the portion of the structure considered. If a multiwythe wall is designed as a composite wall, the compressive strength of each wythe or grouted collar joint should equal or exceed f'_m or f'_{AAC}.

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PART 2: DESIGN REQUIREMENTS

CHAPTER 4
GENERAL ANALYSIS AND DESIGN CONSIDERATIONS

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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 legally adopted building code of which this Code forms a part, with such live load reductions as are permitted in the legally adopted building code. In the absence of a legally adopted building code, or in the absence of design loads in the legally adopted building code, the load provisions of ASCE/SEI 7 shall be used, 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.

COMMENTARY

4.1 — Loading

4.1.2 *Load provisions*
These provisions establish minimum design load requirements. If the design loads specified by the legally adopted building code differ from those of ASCE/SEI 7, the legally adopted building code governs.

4.1.3 *Lateral load resistance*
Interior walls, infill panels, and similar members may not be a part of the lateral-force-resisting system if isolated.

4.1.4 *Load transfer at horizontal connections*
Masonry walls, pilasters, and columns may be connected to horizontal members of the structure and may rely on the latter for lateral support and stability. The mechanism through which the interconnecting forces are transmitted may involve bond, mechanical anchorage, friction, bearing, or a combination thereof. The designer must assure that, regardless of the type of connection, the interacting forces are safely resisted.

~~In flexible frame construction, t~~The relative movement (drift) between floors may generate forces within the members and the connections. This Code requires the effects of these movements to be considered in design.

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4.1.5 *Other effects*
Loads are not the sole source of stresses. The structure may also resist forces from the sources listed. The nature and extent of some of these forces may be greatly influenced by the choice of materials, structural connections, and geometric configuration.

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4.1.6 *Lateral load distribution*

Lateral loads shall be distributed to the structural system in accordance with relative~~member~~ stiffnesses of structural members including horizontal diaphragms and shall comply with the requirements of this section.

4.1.6.1 Flanges of intersecting walls designed in accordance with Section ~~5.1.1.1~~5.2.3 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.

COMMENTARY

4.1.6 *Lateral load distribution*

The design assumptions for masonry buildings include the distribution of forces to the~~use of a~~ lateral-force-resisting system. ~~The distribution of lateral loads to the members of the lateral-force-resisting system is a function of the rigidities of the structural system and of the horizontal diaphragms.~~ Refer to ASCE/SEI 7 for more information about the methods used to distribute load to the lateral force-resisting system. The method of connection at intersecting walls and between walls and floor and roof diaphragms determines if the wall participates in the lateral-force-resisting system. Lateral loads from wind and seismic forces are normally considered to act in the direction of the principal axes of the structure. Lateral loads may cause forces in walls both perpendicular and parallel to the direction of the load. Horizontal torsion can be developed due to eccentricity of the applied load with respect to the center of rigidity. The analysis of lateral load distribution should be in accordance with accepted engineering procedures.

Analysis should also rationally consider the effects of openings in shear walls. The interaction of coupled shear walls is complex and further information may be obtained from Kingsley et al (2014) and Ashour et al (2016). The designer should assess the effects of coupling. The governing behavior mode will depend on whether the configuration of vertical and horizontal oriented wall segments allows the segments to act as an uncoupled or coupled shear wall. Walls may be coupled by concrete slabs, and in this configuration, the coupling effect of the slabs can contribute increased axial forces and moments generated in walls (Kingsley et al (2014) and Ashour et al (2016)). Masonry beams can also couple masonry walls. If the walls are subject to significant lateral displacement, the coupling beams are more likely to fail in shear than meet the deformation demand.

Calculation of the stiffness of shear walls should consider shearing and flexural deformations. A guide for solid shear walls (that is, with no openings) is given in Figure CC-4.1-1. For ungrouted hollow unit shear walls, the use of equivalent solid thickness of wall in calculating web stiffness is acceptable.

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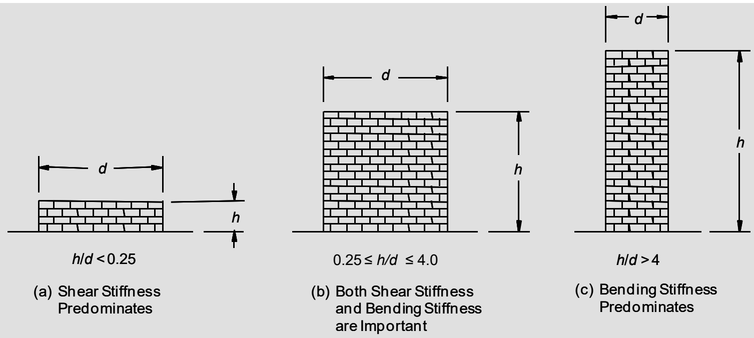

Figure CC-4.1-1 — Shear wall stiffness

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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 *Modulus of elasticity and modulus of rigidity* — The modulus of elasticity and modulus of rigidity shall be taken as the values shown in Table 4.2.2.

Table 4.2.2: Modulus of Elasticity and Modulus of Rigidity

Material	Modulus of Elasticity	Modulus of Rigidity
Steel Reinforcement	E_s = 29,000,000 psi (200,000 MPa)	---
Prestressing Steel[1]	E_{ps} shall be determined by tests or provided by manufacturer	---
Clay Masonry[2]	$E_m = 700 f'_m$	$G = 0.4E_m$
Concrete Masonry[2]	$E_m = 900 f'_m$	$G = 0.4E_m$
Cast Stone Masonry~~[2]~~	~~$E_{mCS} = 57{,}000\sqrt{f'_m}$~~ E_{MCS} shall be determined by tests or provided by manufacturer	--- ~~$G = 0.4E_{mCS}$~~
AAC Masonry	$E_{AAC} = 6500\ (f'_{AAC})^{0.6}$	$G = 0.4\ E_{AAC}$
Grout	$E_g = 500 f'_g$	---

[1] As an alternative for prestressing steel, the modulus of elasticity, E_{ps}, shall be permitted to be taken as 29,000,000 psi (200,000 MPa) for wires and bars and ~~27,560,000 psi (190,000 MPA)~~ 28,000,000 psi (193,000 MPa) for strands.

[2] As an alternative for clay~~,~~ and concrete~~, and cast stone~~ masonry, the modulus of elasticity shall be permitted to be taken equal to the chord modulus 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, and ASTM E111.

COMMENTARY

4.2 — Material properties

4.2.1 *General*

Proper evaluation of the building material movement from all sources is important to masonry design. Clay masonry and concrete masonry may behave quite differently under normal loading and weather conditions. The designer is encouraged to review industry standards for further design information and movement joint locations. Material properties can be determined by appropriate tests of the materials to be used.

4.2.2 *Modulus of elasticity and modulus of rigidity* — This table provides design values for the modulus of elasticity and the modulus of rigidity which are commonly used in the design of masonry. Other modulii may exist.

Prestressing steel — The modulus of elasticity of prestressing steel is often taken equal to 28,000 ksi (193,000 MPa) for design, but can vary and should be verified with the manufacturer.

Clay masonry and concrete masonry — For clay and concrete masonry, the elastic modulus is determined as a function of masonry compressive strength using the relations developed from an extensive survey of modulus data by Wolde-Tinsae et al (1993) and results of a test program by Colville et al (1993). Code values for Em are higher than indicated by a best fit of data relating Em to the compressive strength of masonry. The higher Code values are based on the fact that actual compressive strength significantly exceeds the specified compressive strength of masonry, f'_m, particularly for clay masonry.The Committee decided the most appropriate elastic modulus is the slope of the stress-strain curve below a stress value of 0.33 f'_m. The value of 0.33 f'_m was originally chosen because it was the allowable compressive stress prior to the 2011 Code. Data at the bottom of the stress strain curve may be questionable due to the seating effect of the specimen during the initial loading phase if measurements are made on the testing machine platens. The Committee therefore decided that the most appropriate elastic modulus for design purposes is the chord modulus from a stress value of 5 to 33 percent of the compressive strength of masonry (see Figure CC-4.2-1). The terms chord modulus and secant modulus have been used interchangeably in the past. The chord modulus, as used here, is defined as the slope of a line intersecting the stress-strain curve at two points, neither of which is the origin of the curve.

By using the Code values, the contribution of each wythe to composite action is more accurately accounted for in design calculations than would be the case if the elastic modulus of each part of a composite wall were based on one specified compressive strength of masonry.

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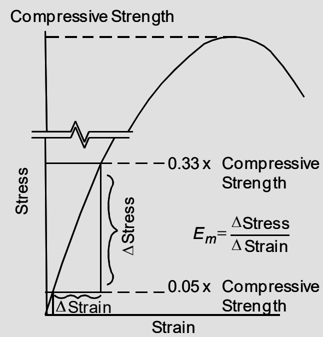

Figure CC-4.2-1 — Chord modulus of elasticity

The relationship between the modulus of rigidity and the modulus of elasticity has historically been given as 0.4 E_m. No experimental evidence exists to support this relationship.

AAC masonry — The modulus of elasticity of autoclaved aerated concrete (AAC) masonry depends almost entirely on the modulus of elasticity of the AAC material itself. The relationship between modulus of elasticity and compressive strength is given in Tanner et al (2005) and Argudo (2003).

Grout — The modulus of elasticity of a grouted assemblage of clay or concrete masonry can usually be taken as a factor multiplied by the specified compressive strength, regardless of the extent of grouting, because the modulus of elasticity of the grout is usually close to that of the clay or concrete masonry. However, grout is usually much stiffer than the AAC material. While it is permissible and conservative to calculate the modulus of elasticity of a grouted assemblage of AAC masonry assuming that the modulus of elasticity of the grout is the same as that of the AAC material, it is also possible to recognize the greater modulus of elasticity of the grout by transforming the cross-sectional area of grout into an equivalent cross-sectional area of AAC, using the modular ratio between the two materials.

Because the inelastic stress-strain behavior of grout is generally similar to that of clay or concrete masonry, calculations of member resistance (whether based on allowable-stress or strength design) usually neglect possible differences in strength between grout and the surrounding masonry. For the same reasons noted above, the stress-strain behavior of grout usually differs considerably from that of the surrounding AAC material. It is possible that these differences in stress-strain behavior could also be considered in calculating member resistances. Research is ongoing to resolve this issue.

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~~*Cast stone* — For cast stone masonry, the elastic modulus is assumed to be similar to concrete, therefore, the equation from ACI 318 is used. Values for specific assemblies can be obtained through testing as outlined in the footnote to Table 4.2.2~~

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4.2.3 *Coefficients of thermal expansion*

The coefficients of thermal expansion shall be taken as the values shown in Table 4.2.3.

Table 4.2.3: Coefficients of Thermal Expansion

Material	Coefficient (k_t)
Clay Masonry	4 x 10^{-6} in./in./°F (7.2 x 10^{-6} mm/mm/°C)
Concrete Masonry and Cast Stone	4.5 x 10^{-6} in./ in./°F (8.1 x 10^{-6} mm/mm/°C)
AAC Masonry	4.5 x 10^{-6} in./ in./°F (8.1 x 10^{-6} mm/mm/°C)
Dimension Stone Masonry	Varies [1]

[1] Specific properties for stone masonry shall be provided by the manufacturer.

4.2.3 *Coefficients of thermal expansion*

Temperature changes cause material expansion and contraction. This material movement is theoretically reversible. These thermal expansion coefficients are slightly higher than mean values for the assemblage (Copeland (1957); Plummer (1962); Grimm (1986)).

Thermal expansion for concrete masonry varies with aggregate type (Copeland (1957); Kalouseb (1954)).

Thermal expansion coefficients are given for AAC masonry in RILEM (1993).

Thermal expansion coefficients for stone masonry will vary depending on the type for stone masonry. Most stone will fall in the range of 4.0 to 5.0 x 10^{-6} in/in/°F (7.2 to 9 mm/mm/°C)

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4.2.4 *Coefficient of moisture expansion for clay masonry*

k_e = 3 x 10^{-4} in./in. (3 x 10^{-4} mm/mm)

4.2.4 *Coefficient of moisture expansion for clay masonry*

Fired clay products expand upon contact with moisture and the material does not return to its original size upon drying (Plummer (1962); Grimm (1986)). This is a long-term expansion as clay particles react with atmospheric moisture. Continued moisture expansion of clay masonry units has been reported for 7½ years (Smith (1973)). Moisture expansion is not a design consideration for concrete masonry.

4.2.5 *Coefficients of shrinkage*

4.2.5.1 *Concrete masonry and cast stone masory*

$k_m = 0.5\ s_l$

4.2.5.2 *AAC masonry*

$k_m = 0.8\ \varepsilon_{cs}/100$

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

4.2.5 *Coefficients of shrinkage*

4.2.5.1 *Concrete masonry* — Concrete masonry is a cement-based material that shrinks due to moisture loss and carbonation (Kalouseb (1954)). The total linear drying shrinkage is determined in accordance with ASTM C426. The maximum shrinkage allowed by ASTM specifications for concrete masonry units (for example, ASTM C90), other than calcium silicate units, is 0.065%. Further design guidance for estimating the shrinkage due to moisture loss and carbonation is available (NCMA TEK 10-1A (2005); NCMA TEK 10-~~2C~~2D (2010); NCMA TEK 10-3 (2003); NCMA TEK 18-2C (2014)). The shrinkage of clay masonry is negligible.

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4.2.5.2 *AAC masonry* — At time of production, AAC masonry typically has a moisture content of about 30%. That value typically decreases to 15% or less within two to three months, regardless of ambient relative humidity. This process can take place during construction or prior to delivery. ASTM C1693 evaluates AAC material characteristics at moisture contents between 5% and 15%, a range that typifies AAC in service. The shrinkage coefficient of this section reflects the change in strain likely to be encountered within the extremes of moisture content typically encountered in service.

4.2.6 *Coefficients of creep*

The coefficients of creep shall be taken as the values shown in Table 4.2.6.

Table 4.2.6: Coefficients of Creep

Material	Coefficient (k_c)
Clay Masonry	0.7 x 10^{-7}, per psi (0.1 x 10^{-4}, per MPa)
Concrete Masonry	42.5 x 10^{-7}, per psi (0.36 x 10^{-4}, per MPa)
AAC Masonry	5.0 x 10^{-7}, per psi (0.72 x 10^{-4}, per MPa)

4.2.6 *Coefficients of creep*

When continuously stressed, these materials gradually deform in the direction of stress application. This movement is referred to as creep and is load and time dependent (Kalouseb (1954); Lenczner and Salahuddin (1976); RILEM (1993)). The values given are maximum values.

4.2.7 *Prestressing steel*

For prestressing steels not specifically listed in ASTM A416/A416M, A421/A421M, or A722/A722M, tensile strength and relaxation losses shall be determined by tests.

4.2.7 *Prestressing steel*

The material and section properties of prestressing steels may vary with each manufacturer. Most significant for design are the prestressing tendon's cross section, modulus of elasticity, tensile strength, and stress-relaxation properties. Values for these properties for various manufacturers' wire, strand, and bar systems are given elsewhere (PTI (2006)). Stress-strain characteristics and stress-relaxation properties of prestressing steels must be determined by test, because these properties may vary between different steel forms (bar, wire, or strand) and types (mild, high strength, or stainless). See also Commentary Section 4.2.2.

4.3 — Specified Compressive Strength

The specified compressive strength of masonry and grout shall meet the requirements of Table 4.3.1.

4.3 — Specified Compressive Strength

Most masonry research, including the design criteria based on TCCMaR research (Noland and Kingsley (1995)), has been conducted on structural masonry components having compressive strength in the range of 1,500 to 4,000 psi (10.34 to 27.58 MPa) for concrete masonry and 1,500 to 6,000 psi (10.34 to 41.37 MPa) for clay masonry. Thus, the upper limits given represent the upper values that were tested in the research.

Research (Varela et al (2006); Tanner et al (2005a)), Tanner et al (2005b); Argudo (2003)) has been conducted on structural components of AAC masonry with a compressive strength of 290 to 1,500 psi (2.0 to 10.3 MPa). Design criteria are based on these research results.

The code does not explicitly stipulate a minimum specified compressive strength for application with its design provisions. Compliance with the material requirements of TMS 602 implicitly establish a minimum masonry compressive strength. Care should be used when applying these provisions to materials and assemblies that do not conform to the requirements of TMS 602.

Because most empirically derived design equations calculate nominal strength as a function of the specified compressive strength of the masonry, the specified compressive strength of the grout is required to be at least equal to the specified compressive strength for concrete masonry. This requirement is an attempt to ensure that where the grout compressive strength controls the design (such as anchors embedded in grout), the nominal strength will not be affected. The limitation on the maximum grout compressive strength is due to the lack of available research using higher material strengths.

Due to the hydrophilic nature of AAC masonry, care should be taken to control grout shrinkage by pre-wetting cells to be grouted or by using other means, such as non-shrink admixtures. Bond between grout and AAC units is equivalent to bond between grout and other masonry units (Tanner et al (2005a), Tanner et al (2005b); Argudo (2003)).

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Table 4.3.1: Specified Compressive Strength Requirements

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Type of masonry	Specified compressive strength of masonry	Specified compressive strength of grout
Concrete masonry	$f'_m \le$ 4,000 psi (27.58 MPa)	$f'_g \ge f'_m \le$ 5,000 psi (34.47 MPa)
Clay masonry	$f'_m \le$ 6,000 psi (41.37 MPa)	$f'_g \le$ 6,000 psi (41.37 MPa)
AAC masonry	$f'_{AAC} \ge$ 290 psi (2.0 MPa)	2,000 psi (13.8 MPa) $\le f'_g \le$ 5,000 psi (34.47 MPa)

~~4.3~~4.4 — Section properties

~~4.3~~4.4.1 *Stress calculations*

~~4.3~~4.4.1.1 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~~4.4.1.2 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~~4.4 — Section properties

~~4.3~~4.4.1 *Stress calculations*

Minimum net section is often difficult to establish in ungrouted hollow unit masonry. The minimum net section may not be the same in the vertical and horizontal directions.

For masonry of ungrouted hollow units, laid in face shell mortar bedding, the minimum cross-sectional area in both directions may conservatively be based on the minimum face-shell thickness (NCMA TEK 14-1B (2007)).

Solid clay masonry units are permitted to have coring up to a maximum of 25 percent of their gross cross-sectional area. For such units, the net cross-sectional area may be taken as equal to the gross cross-sectional area, except as provided in Section ~~8.1.4~~8.1.5.2(c) or 9.1.7.2(c) for masonry headers. Several conditions of net area are shown in Figure CC-~~4.3~~4.4-1.

When the elastic properties of the materials used in members designed for composite action differ, equal strains produce different levels of stresses in the components. To calculate these stresses, a transformed section with respect to the axis of resistance is considered. The resulting stresses developed in each fiber are related to the actual stresses by the ratio E_1 / E_x between the modulus of elasticity, E_1, of the most deformable material in the member and the modulus of elasticity, E_x, of the materials in the fiber considered. Thus, to obtain the transformed section, fibers of the actual section are conceptually widened by the ratio E_x / E_1. Stresses calculated based on the section properties of the transformed section, with respect to the axis of resistance considered, are then multiplied by E_x / E_1 to obtain actual stresses.

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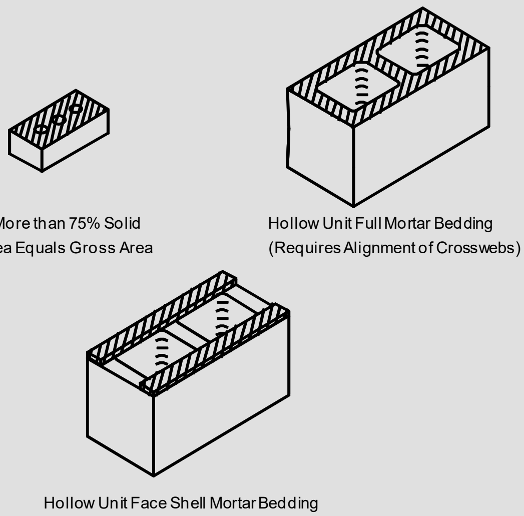

Figure CC-~~4.3~~4.4-1 — Net cross-sectional areas

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~~4.3~~4.4.2 *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~~4.4.3 *Radius of gyration*

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

COMMENTARY

~~4.3~~4.4.2 *Stiffness*

Stiffness is a function of the extent of cracking. Because unreinforced masonry is designed assuming it is uncracked, Code equations for design of unreinforced masonry are based on the member's uncracked moment of inertia and ignoring the effects of reinforcement, if present. For reinforced masonry, more accurate estimates may result if stiffness approximations are based on the cracked section.

The section properties of masonry members may vary from point to point. For example, in a single-wythe concrete masonry wall made of hollow ungrouted units, the cross-sectional area varies through the unit height. Also, the distribution of material varies along the length of the wall or unit. For stiffness calculations, an average value of the appropriate section property (cross-sectional area or moment of inertia) is considered adequate for design. The average net cross-sectional area of the member would in turn be based on average net cross-sectional area values of the masonry units and the mortar joints composing the member.

~~4.3~~4.4.3 *Radius of gyration*

The radius of gyration is the square root of the ratio of bending moment of inertia to cross-sectional area. Because stiffness is based on the average net cross-sectional area of the member considered, this same area should be used in the calculation of radius of gyration. To simplify the calculation of radius of gyration, tabulated section properties for walls consisting of hollow units with a

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~~4.3~~4.4.4 *Bearing area*

The bearing area, A_{br}, for concentrated loads shall be calculated as the lesser of the following:

$$A_{br} = A_1\sqrt{A_2 / A_1} \qquad \text{(Equation 4-1)}$$

$$A_{br} = 2A_1 \qquad \text{(Equation 4-2)}$$

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.

~~4.3~~4.4.5 *Net shear area*

The net shear area, A_{nv}, for reinforced masonry members shall be determined from Table ~~4.3~~4.4.5

COMMENTARY

variety of grout configurations are available in NCMA TEK 14-1B (2007) and BIA TN 3B (1993).

~~4.3~~4.4.4 *Bearing area*

When the supporting masonry area, A_2, is larger on all sides than the loaded area, A_1, this Code allows distribution of concentrated loads over a bearing area A_{br}, larger than A_1. The area A_2 is determined as illustrated in Figure CC-~~4.3~~4.4-2. This is permissible because the confinement of the bearing area by surrounding masonry increases the bearing capacity of the masonry under the concentrated loads. When the edge of the loaded area, A_1, coincides with the face or edge of the masonry, the area A_2 is equal to the loaded area A_1.

~~4.3~~4.4.5 *Net shear area*

The equations for shear in this Code were developed using data from shear wall tests. The net shear area was based on the full length of the shear wall, d_v (Shing et al (1990)). For partially grouted shear walls, the net shear area includes the face shell mortar bedding and grouted cells. For flanged shear walls, the net shear area is only the segment of the shear wall that lies parallel to the direction of applied shear. For beams, the Code shear equations assume the shear area is based on the depth to the centroid of the flexural reinforcement, d (Sarhat and Sherwood (2010)). A net shear area based on d is also used for the net shear area of walls loaded out-of-plane.

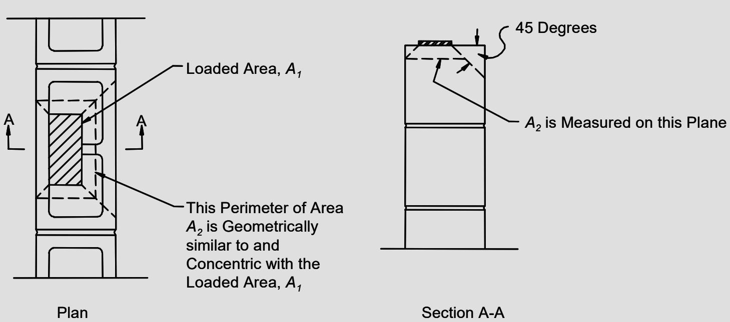
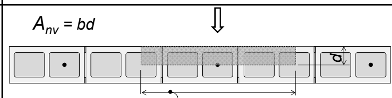
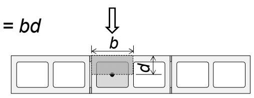
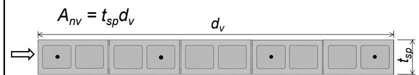
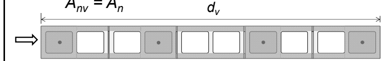
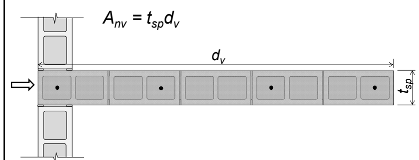
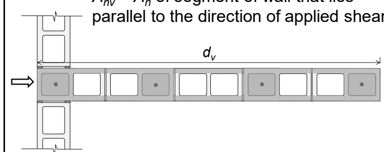
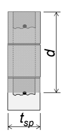
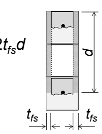


Figure CC-~~4.3~~4.4-2 — Bearing areas

Table ~~4.3~~4.4.5: Net Shear Area for Reinforced Masonry Members

Loading Direction / Member Type	Fully Grouted	Partially Grouted
Out-of-Plane / Wall	$A_{nv} = bd$  b = effective compression width (Section 5.1.2)	$A_{nv} = bd$ 
In-plane / Planar Shear Wall	$A_{nv} = t_{sp}d_v$ 	$A_{nv} = A_n$  A_n = net cross-sectional area
In-plane / Flanged Shear Wall	$A_{nv} = t_{sp}d_v$ 	$A_{nv} = A_n$ of segment of wall that lies parallel to the direction of applied shear 
Beams	$A_{nv} = t_{sp}d$ 	$A_{nv} = 2t_{fs}d$ 

TMS 402 CODE

~~4.4~~4.5 — 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~~4.6 — Deflection of beams supporting unreinforced masonry

The calculated deflection of beams of any material providing vertical support to masonry designed in accordance with Section 8.2, Section 9.2, Section 11.2, or Chapter 15 shall not exceed $\ell/600$ under allowable stress level dead plus live loads.

COMMENTARY

~~4.4~~4.5 — Connection to structural frames

Differential movements between masonry and a structural frame may occur due to the following:

1) Temperature increase or decrease of either the structural frame or the masonry wall.
2) Moisture and freezing expansion of brick or shrinkage of concrete block walls.
3) Elastic shortening of columns from axial loads, shrinkage, or creep.
4) Deflection of supporting beams.
5) Sidesway in multiple-story buildings.
6) Foundation movement.

The designer should consider differential movements and the forces resulting from their restraint. The type of connection chosen should transfer only the loads planned. While load transfer usually involves masonry attached to structural members, such as beams or columns, the connection of nonstructural components, such as door and window frames, to masonry members should also be addressed.

Structural frames and bracing should not be infilled with masonry to increase resistance to in-plane lateral forces without considering the differential movements listed above.

~~4.5~~4.6 — Deflection of beams supporting unreinforced masonry

The deflection limits apply to beams and lintels of any material that supports unreinforced masonry. The deflection requirements may also be applicable to supported reinforced masonry that has vertical reinforcement only.

The deflection limit of $\ell/600$ should prevent long-term visible deflections and serviceability problems. In most cases, deflections of approximately twice this amount, or $\ell/300$, are required before the deflection becomes visible (Galambos and Ellingwood (1986)). This deflection limit is for immediate deflections. Creep will cause additional long-term deflections. A larger deflection limit of $\ell/480$ has been used when considering long-term deflections (CSA (2014)).

TMS 402 CODE

~~4.6~~4.7 — 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 48 in. (1219 mm) on center in horizontal mortar joints or in bond beams.

COMMENTARY

~~4.6~~4.7 — Masonry not laid in running bond

The requirements for masonry laid in running bond are shown in Figure CC-~~4.6~~4.7-1. The amount of horizontal reinforcement required in masonry not laid in running bond is a prescriptive amount to provide continuity across the head joints. Because lateral loads are reversible, reinforcement should either be centered in the member thickness by placement in the center of a bond beam, or should be symmetrically located by placing multiple bars or deformed wires in a bond beam or by using joint reinforcement or deformed wires in the mortar bed along each face shell. This reinforcement can be also used to resist load.

Although continuity across head joints in masonry not laid in running bond is a concern for AAC masonry as well as masonry of clay or concrete, the use of horizontal reinforcement to enhance continuity in AAC masonry is generally practical only by the use of bond beams.

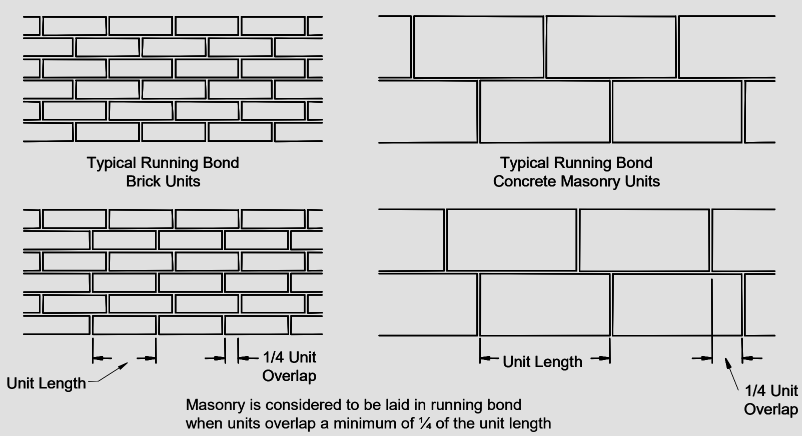


Figure CC-~~4.6~~4.7-1 — Running bond masonry

TMS 402 CODE

~~4.7~~4.8 — 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.

~~4.7~~4.8.1 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.

~~4.7~~4.8.2 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.

~~4.7~~4.8.3 Vertical conduits, pipes, or sleeves placed in masonry columns or pilasters shall not displace more than 2 percent of the net cross section.

~~4.7~~4.8.4 Pipes shall not be embedded in masonry, unless properly isolated from the masonry, when:

(a) Containing liquid, gas, or vapors at temperature higher than 150º F (66ºC).

(b) Under pressure in excess of 55 psi (379 kPa).

(c) Containing water or other liquids subject to freezing.

~~4.7~~4.8.5 Placement of conduits, pipes, and sleeves in unfilled cores of hollow unit masonry shall be permitted and shall not be considered embedded.

~~4.7~~4.8.6 Aluminum shall not be used in masonry unless it is effectively coated or otherwise isolated.

COMMENTARY

~~4.7~~4.8 — Embedded conduits, pipes, and sleeves

~~**4.7.1** *Embedded conduits, pipes, and sleeves*~~

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~~4.7~~4.8.1 Conduits, pipes, and sleeves not harmful to mortar and grout may be embedded within the masonry, but the masonry member strength should not be less than that required by design. Effects of reduction in section properties in the areas of conduit, pipe, or sleeve embedment should be considered.

For the integrity of the structure, conduit and pipe fittings within the masonry should be carefully positioned and assembled. The coupling size should be considered when determining sleeve size.

Pipes and conduits placed in masonry, whether surrounded by mortar or grout or placed in unfilled spaces, need to allow unrestrained movement.

~~4.7~~4.8.6 Aluminum reacts with ions, and may also react electrolytically with steel, causing cracking, spalling of the masonry, or both. Aluminum electrical conduits present a special problem because stray electric current accelerates the adverse reaction.

CHAPTER 5
STRUCTURAL MEMBERS

TMS 402 CODE

5.1 — ~~Masonry assemblies~~General

5.1.~~3~~1 *Concentrated loads*

5.1.~~3~~1.1 Concentrated loads shall not be distributed over a length greater than either (a) or (b), but not to exceed the center-to-center distance between concentrated loads:

(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) Where a concentrated load is applied adjacent to an opening, or end of wall, the length of bearing area plus the length determined by considering the concentrated load to be dispersed along a 3 vertical: 1 horizontal line. The dispersion on each side shall terminate independently at half the wall height, a movement joint, the end of the wall, or an opening, whichever provides the smallest length.

5.1.~~3~~1.2 For assemblies not laid in running bond, concentrated loads shall not be distributed across head joints. Where concentrated loads acting on such assemblies are applied to a bond beam, the concentrated load shall be permitted to be distributed through the bond beam, but shall not be distributed across head joints below the bond beams.

COMMENTARY

5.1 — ~~Masonry assemblies~~General

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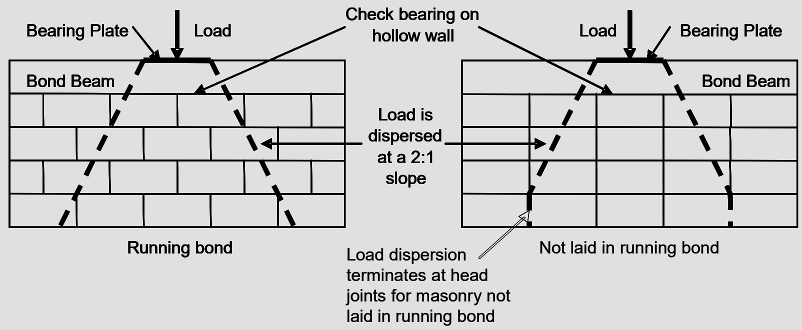
5.1.~~3~~1 *Concentrated loads*

Arora (1988) reports the results of tests of a wide variety of specimens under concentrated loads, including AAC masonry, concrete block masonry, and clay brick masonry specimens. Arora (1988) suggests that a concentrated load can be distributed at a 2:1 slope, terminating at half the wall height. Tests on the load dispersion through a bond beam on top of hollow masonry reported in Page and Shrive (1987) resulted in an angle from the horizontal of 59 degrees for a 1-course CMU bond beam, 65 degrees for a 2-course CMU bond beam, and 58 degrees for a 2-course clay bond beam, or approximately a 2:1 slope. For simplicity in design, a 2:1 slope is used for all cases of load dispersion of a concentrated load.

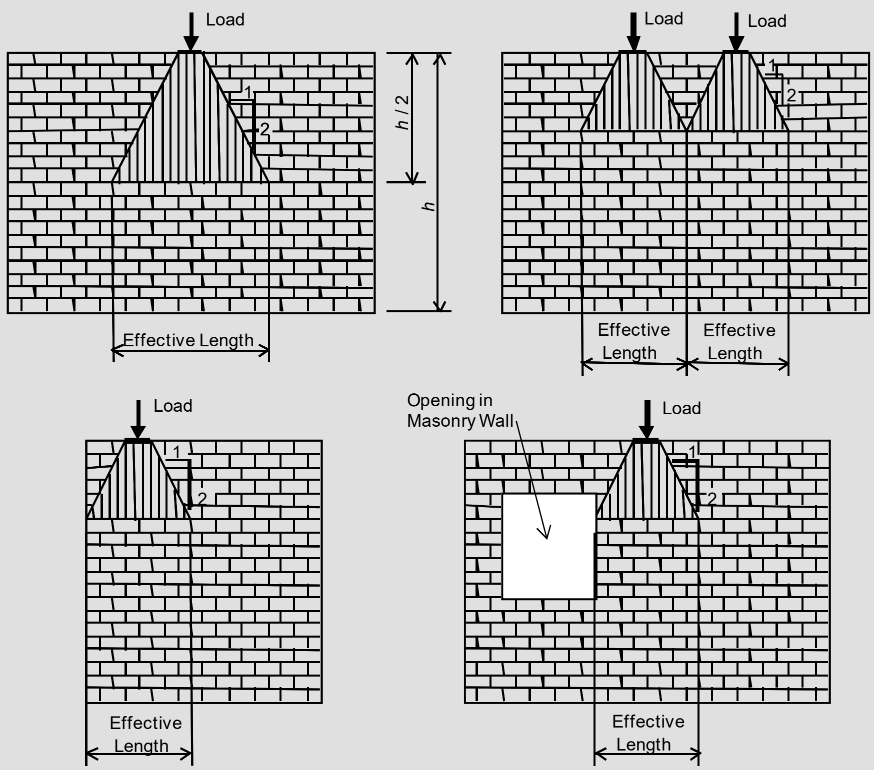
(a) Code provisions are illustrated in Figure CC-5.1-~~5~~1. Figure CC-5.1-~~5~~1a illustrates the dispersion of a concentrated load through a bond beam. A hollow wall would be checked for bearing under the bond beam using the length at this location. Figure CC-5.1-5~~b~~1 illustrates the dispersion of a concentrated load in the wall. The length of the wall effective under the concentrated load is used for checking the wall's load resistance. A wall may have to be checked at several locations, such as under a bond beam and at midheight.

(b) Where a concentrated load is applied to a masonry wall adjacent to the edge of an opening or wall edge, stopping the 2 to 1 dispersion at the edge of the wall can significantly underestimate the length of the wall resisting the load. The testing by Arora (1988) showed that the load at the wall edge can be distributed assuming a single sided distribution of 70 degrees. For this condition, the dispersion angle was set at the 3:1 value to account for a wider range of wall area to bearing area configurations and the lower confinement provided by the wall near an edge. This steeper dispersion will continue away from the opening up to ½ the height of the masonry below the load ($H_{below\ load}$) so the dispersions can be truncated independently on each side of the bearing. Figure CC-5.1-~~5~~1c illustrates the dispersion of a concentrated load in the wall near an opening edge.

COMMENTARY



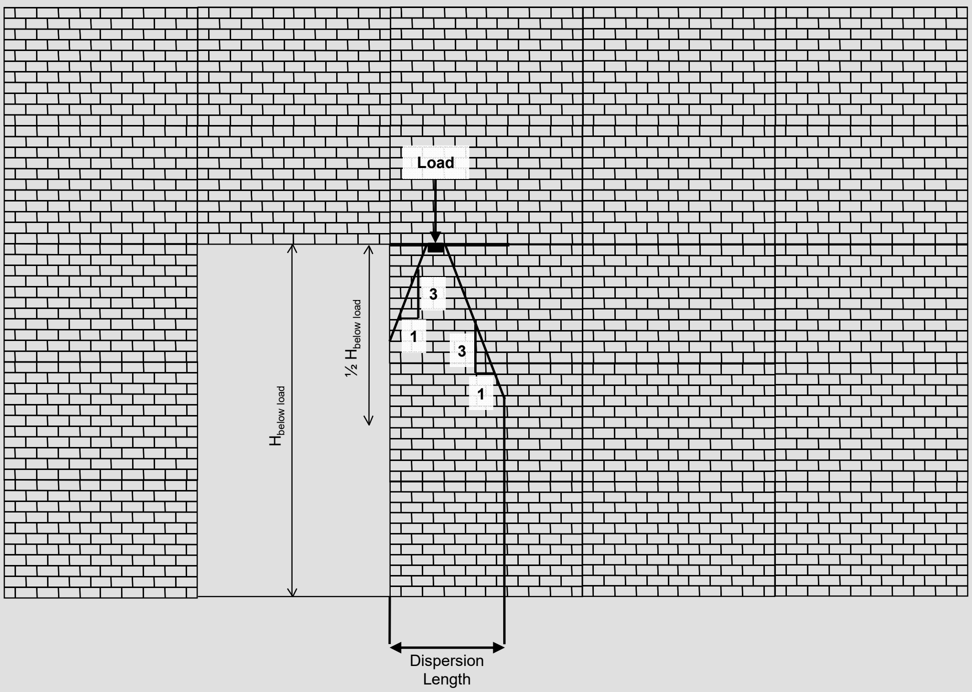
(a) Distribution of concentrated load through bond beam



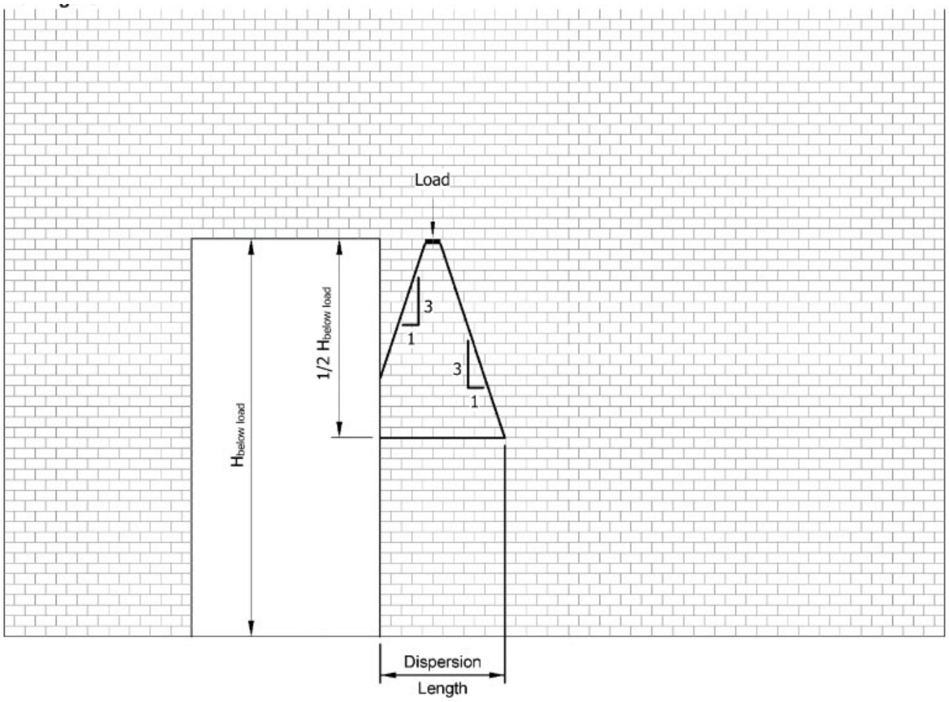
(b) Distribution of concentrated load in wall

Figure CC-5.1-~~5~~1. Distribution of concentrated loads

COMMENTARY



(c) Distribution of concentrated load in a wall adjacent to an opening



(c) Distribution of concentrated load in a wall adjacent to an opening

Figure CC-5.1-~~5~~1 (Continued). Distribution of concentrated loads

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5.1.2 *Effective compressive width*

5.1.2.1 For masonry not laid in running bond and having bond beams spaced not more than 48 in. (1219 mm) center-to-center, and for masonry laid in running bond, the width of the compression area used to calculate member capacity shall not exceed the least of:

(a) Center-to-center spacing of the reinforcement.

(b) Six multiplied by the nominal wall thickness.

(c) 72 in. (1829 mm).

5.1.2.2 For masonry not laid in running bond and having bond beams spaced more than 48 in. (1219 mm) center-to-center, the width of the compression area used to calculate member capacity shall not exceed the length of the masonry unit.

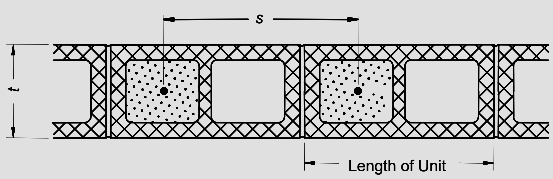
5.1.2 *Effective compressive width*

The effective width of the compressive area for each reinforcing bar or deformed wire must be established. Figure CC-5.1-24 depicts the limits for the conditions stated. Limited research (Dickey and MacIntosh (1971)) is available on this subject.

The limited ability of head joints to transfer stress when masonry is not laid in running bond is recognized by the requirements for bond beams. Open end masonry units that are fully grouted are assumed to transfer stress across the head joints even when not laid in running bond.

The center-to-center reinforcement spacing maximum is a limit to keep from overlapping areas of compressive stress. The 72-in. (1829-mm) maximum is an empirical choice of the Committee.

COMMENTARY



For masonry not laid in running bond with bond beams spaced less than or equal to 48 in. (1219 mm) center-to-center and running bond masonry, *b* equals the lesser of:

$b = s$
$b = 6t$
b = 72 in. (1829 mm)

For masonry not laid in running bond with bond beams spaced greater than 48 in. (1219 mm) center-to-center, *b* equals the lesser of:

$b = s$
b = length of unit

Figure CC-5.1-24 — Width of compression area

TMS 402 CODE

5.1.~~4~~3 *Multiwythe masonry*

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~~3.2 or 5.1.~~4~~3.3.

5.1.~~4~~3.1 The provisions of Sections 5.1.~~4~~3.2, and 5.1.~~4~~3.3 shall not apply to AAC masonry units, masonry veneer, and glass masonry units.

5.1.~~4~~3.2 *Composite action*

5.1.~~4~~3.2.1 Multiwythe masonry designed for composite action shall have wythes connected by either:

(a) masonry headers, or

(b) collar joints and wall ties.

5.1.~~4~~3.2.2 Headers used to bond adjacent wythes shall meet the requirements of either Section ~~8.1.4~~8.1.5.2 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 3 in. (76.2 mm) in each wythe.

5.1.~~4~~3.2.3 Wythes not bonded by headers shall meet the requirements of either Section ~~8.1.4~~8.1.5.2 or Section 9.1.7.2 and shall be bonded by non-adjustable wall ties according to Table 5.1.~~4~~3.2.3.

COMMENTARY

5.1.~~4~~3.2 *Composite action* — Multiwythe masonry acts monolithically if sufficient shear transfer can occur across the interface between the wythes. See Figure CC-5.1-~~6~~3. Shear transfer is achieved with headers that connect the wythes or with collar joints. When collar joints are relied upon to transfer shear, wall ties are required to ensure structural integrity of the collar joint. Composite action requires that the shear stresses occurring at the interfaces are within the limits prescribed.

Composite masonry walls generally consist of brick-to-brick, block-to-block, or brick-to-block wythes. The collar joint thickness ranges from $^3/_8$ to 4 in. (9.5 to 102 mm). The joint may contain either vertical or horizontal reinforcement, or reinforcement may be placed in either the brick or block wythe. Composite masonry is particularly advantageous for resisting high loads, both in-plane and out-of-plane.

5.1.~~4~~3.2.2 Requirements for masonry headers (Figure CC-5.1-~~7~~4) are empirical and taken from prior codes. The net area of the header should be used in calculating the stress even if a solid unit, which is allowed to have up to 25 percent coring, is used. Headers are less ductile than metal wall ties, making differential movement a critical issue, particularly when headers are used to bond two dissimilar masonry materials together such as clay masonry and concrete masonry. Differential movement can shear the headers, eliminating composite action.

5.1.~~4~~3.2.3 The required size, number, and spacing of wall ties in composite masonry, shown in Figure CC-5.1-~~8~~5, has been determined from past experience. The limitation of Z-ties to masonry of other than hollow units is also based on past experience.

COMMENTARY

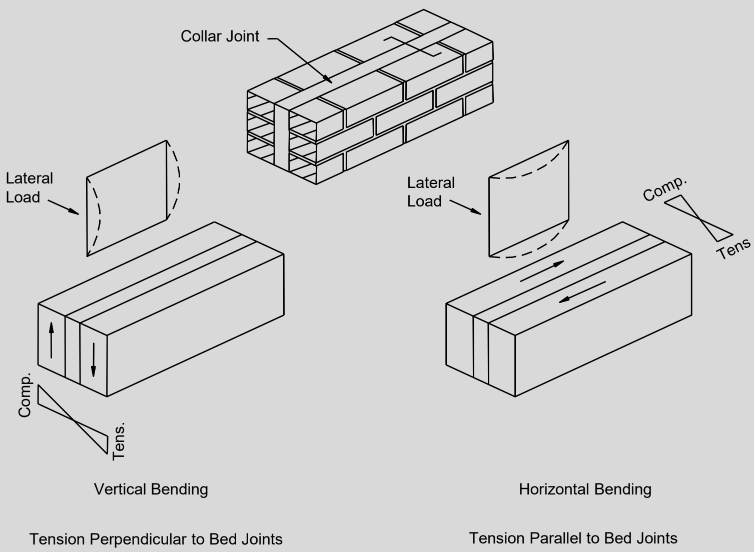


Figure CC-5.1-~~6~~3 — Stress distribution in composite masonry

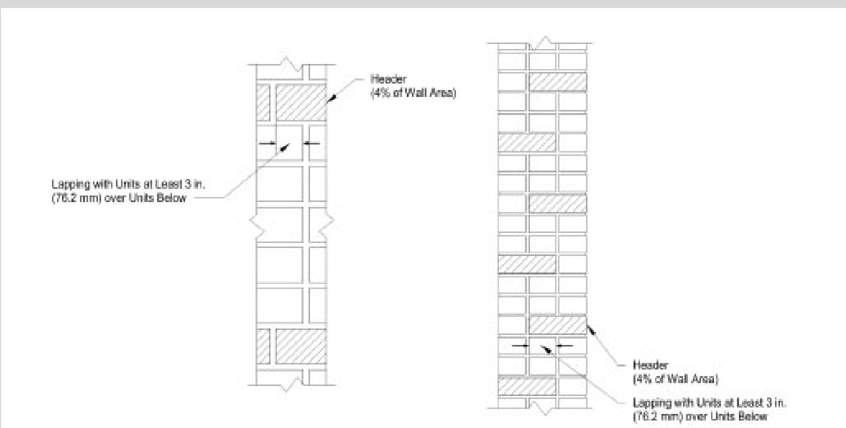


Figure CC-5.1-~~7~~4 — Cross section of wall elevations

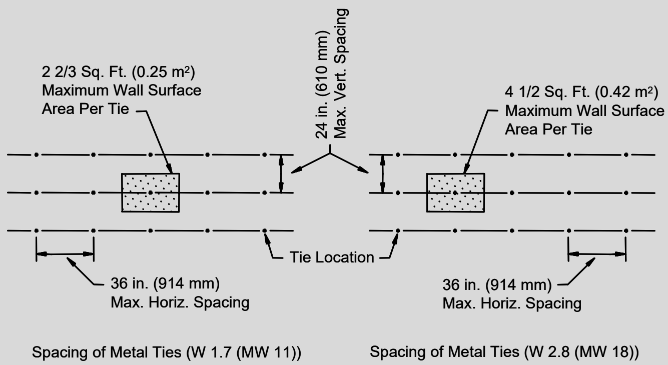


Figure CC-5.1-~~8~~5 — Required wall tie spacing for composite masonry

Table 5.1.~~4~~3.2.3: Maximum Spacing and Wall Area for Wall Ties in Multiwythe Masonry Designed per Chapter 8, Chapter 9, or Chapter 10

Masonry Design Approach	**Tie Type and Size (cavity drips not permitted)**			
	Adjustable Wire[1]	**Non-Adjustable Wire**		**Joint Reinforcement**
	W2.8 (MW18)	W1.7 (MW11)	W2.8 (MW18)	W1.7 (MW11) minimum wire
Masonry Designed for Non-Composite Action				
Maximum Wall Area per Tie	1.77 ft^2 (0.16 m^2)	2.67 ft^2 (0.25 m^2)	4.50 ft^2 (0.42 m^2)	Same as non-adjustable unit ties of same wire size
Maximum Horizontal Spacing	16 in. (406 mm)	36 in. (914 mm)	36 in. (914 mm)	16 in. (406 mm)
Maximum Vertical Spacing	16 in. (406 mm)	24 in. (610 mm)	24 in. (610 mm)	24 in. (610 mm)
Masonry Designed for Composite Action				
Maximum Wall Area per Tie	Not permitted	2.67 ft^2 (0.25 m^2)	4.50 ft^2 (0.42 m^2)	Same as non-adjustable unit ties of same wire size
Maximum Horizontal Spacing	Not permitted	36 in. (914 mm)	36 in. (914 mm)	16 in. (406 mm)
Maximum Vertical Spacing	Not permitted	24 in. (610 mm)	24 in. (610 mm)	24 in. (610 mm)

[1] A minimum of two pintle legs and two horizontal legs are required.

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5.1.~~4~~3.3 *Non-composite action* — The design of multiwythe masonry for non-composite action shall comply with Sections 5.1.~~4~~3.3.1 and 5.1.~~4~~3.3.2.

5.1.~~4~~3.3.1 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) Cavities 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 shall be distributed to each wythe in proportion to its relative stiffness.

(c) Lateral loads acting parallel to the plane of the masonry 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 shall be resisted by all wythes in proportion to their relative flexural stiffnesses.

(e) Specified distances between wythes shall not exceed 4.5 in. (114 mm) unless a detailed tie analysis is performed.

5.1.~~4~~3.3.2 Wythes of masonry designed for non-composite action shall be connected by ties meeting the requirements of Section 5.1.~~4~~3.2.3 or by adjustable wall ties according to Table 5.1.~~4~~3.2.3. Where the cross wires of joint reinforcement are used as wall ties, the joint reinforcement shall be ladder-type or tab-type and shall conform with spacing requirements of Table 5.1.~~4~~3.2.3. Wall ties shall be without cavity drips.

COMMENTARY

5.1.~~4~~3.3 *Non-composite action* — Multiwythe masonry may be constructed so that each wythe is separated from the others by a space that may be crossed only by wall ties. The wall ties force compatible lateral deflection, but no composite action exists in the design.

5.1.~~4~~3.3.1 Weak-axis bending moments caused by either gravity loads or lateral loads are assumed to be distributed to each wythe in proportion to its relative stiffness. See Figure CC-5.1-~~9~~6 for stress distribution in non-composite masonry. ~~In non-composite masonry, the plane of the masonry is the plane of the space between wythes.~~ Loads due to supported horizontal members are to be resisted by the wythe closest to center of span as a result of the deflection of the horizontal member and subsequent rotation at the support. See Figure CC-5.1-7.

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In non-composite masonry, this Code limits the thickness of the cavity to 4.5 in. (114 mm) to assure adequate performance. If cavity width exceeds 4.5 in. (114 mm), the wall ties must be designed to resist the loads imposed upon them based on a rational analysis that takes into account buckling, tension, pullout, and load distribution.

The NCMA and Canadian Standards Association (NCMA TEK 12-1B (2011); CSA (2014)) have recommendations for use in the design of ties for masonry with wide cavities.

5.1.~~4~~3.3.2 The required size, number, and spacing of metal wall ties in non-composite masonry (Figure CC-5.1-~~8~~5) have been determined from past experience. Requirements for adjustable wall ties are shown in Figure CC-5.1-~~10~~8. They are based on the results in IIT (1963). Ladder-type or tab-type joint reinforcement is required because truss-type joint reinforcement restricts in-plane differential movement between wythes. However, the use of ties with drips (bends in ties to prevent moisture migration) has been eliminated because of their reduced strength.

COMMENTARY

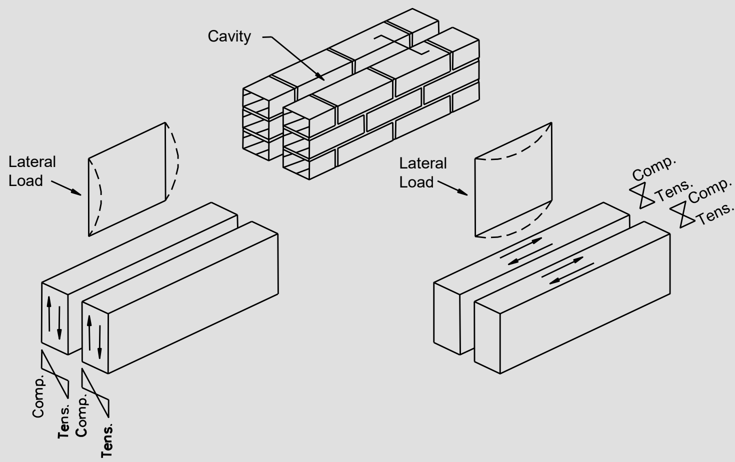

Figure CC-5.1-96 — Stress distribution in non-composite multiwythe masonry

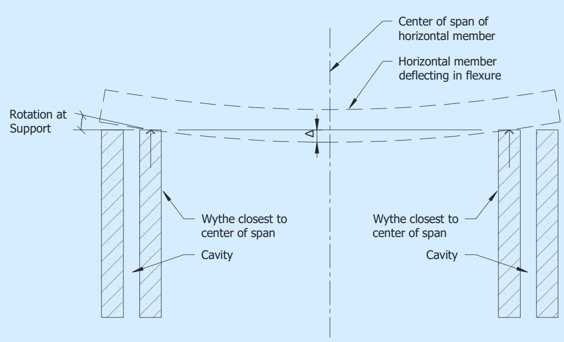

Figure CC-5.1-7 — Loading of inner wythe due to flexure in supported horizontal member

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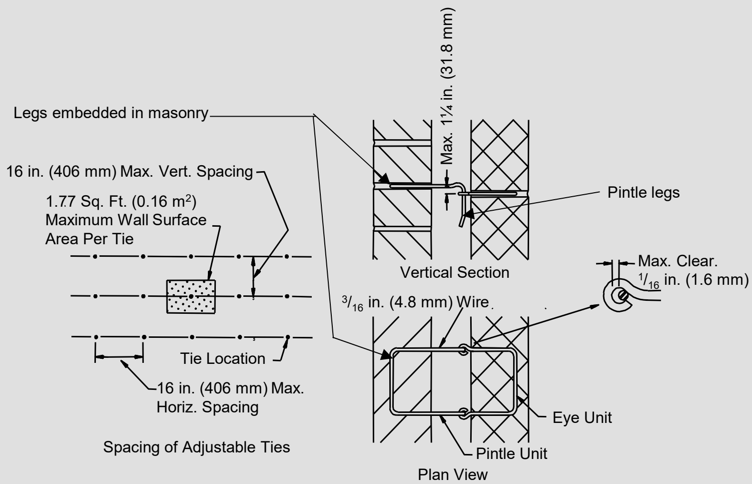

Figure CC-5.1-~~10~~8 — Required spacing of adjustable wall ties for non-composite masonry

~~5.1.1~~5.2 Walls~~ intersectons~~

~~5.1.1 *Wall intersections*~~

Masonry walls that intersect and require ~~depending upon one another for~~ lateral support from one another~~,~~ or ~~upon~~from pilasters within those walls, shall be anchored or bonded at locations where they meet or intersect per Section ~~5.1.1.1~~5.2.2 or ~~5.1.1.2~~5.2.3. Masonry walls that intersect and do not require lateral support from other walls or pilasters within those walls shall be designed in accordance with Section ~~5.1.1.3~~5.2.1

5.2 Walls

~~5.1.1 *Wall intersections*~~

Wall intersections may be designed and detailed as fully composite walls, as laterally supported walls, or as structurally independent walls in accordance with Sections ~~5.1.1~~5.2.1, ~~5.1.1~~5.2.2, and ~~5.1.1~~5.2.3. Acceptable methods of detailing laterally supported walls may include the use of mesh ties, joint reinforcement, or anchors capable of transferring lateral loads only at the interface of laterally supported walls.

Movement joints at structurally independent walls should be sized to prevent force transfer when the walls laterally deform.

Connections of webs to flanges of walls may be accomplished by running bond, metal connectors, or bond beams. Achieving stress transfer at a T intersection with running bond only is difficult due to constructability and modularity of the units. A running bond connection is shown in Figure CC-~~5.1-1~~5.2-1 with a "T" geometry over their intersection.

The alternate method, using metal strap connectors, is shown in Figure CC-~~5.1-2~~5.2-2. Bond beams, shown in Figure CC-~~5.1-~~5.2-33, are the third means of connecting webs to flanges.

When the flanges are connected at the intersection, they are required to be included in the design.

The effective width of the flange for compression and unreinforced masonry in flexural tension is based on shear-lag effects and is a traditional requirement. The effective width of the flange for reinforced masonry in flexural tension is based on the experimental and analytical work of He and Priestley (1992). They showed that the shear-lag effects are significant for uncracked walls, but become less severe after cracking. He and Priestley (1992) proposed that the effective width of the flange be determined as:

$$l_{ef} = \begin{cases} l_f & l_f / h \le 1.5 \\ 0.75h + 0.5 l_f & 1.5 < l_f / h \le 3.5 \\ 2.5h & l_f / h > 3.5 \end{cases}$$

where l_{ef} is the effective flange width, l_f is the width of the flange, and h is height of the wall. These equations can result in effective flange widths greater than 1.5 times the height of the wall. However, a limit of the effective flange width of 1.5 times the wall height, or ¾ of the wall height on either side of the web, is provided in the Code. This limit was chosen because the testing by He and Priestley (1992) was limited to a flange width of 1.4 times the wall height. Designers are cautioned that longitudinal reinforcement just outside the effective flange width specified by the Code can affect the ductility and behavior of the wall. Any participation by the reinforcement in resisting the load can lead to other, more brittle, failure modes such as shear or crushing of the compression toe.

TMS 402 Code and Commentary, C-61

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Commented [PJS72]: 20-SM-020B

Commented [PJS73]: 20-SM-022C

~~5.1.1.3~~5.2.1 *Design of independent walls*

~~5.1.1.3.1~~5.2.1.1 Wall intersections shall be designed and detailed such that no forces are transferred between walls.

~~5.1.1.5~~5.2.2 *Design of lateral supports for walls, without composite action at the intersections*

Masonry walls depending upon intersecting masonry ~~supporting~~ walls or pilasters for lateral support, without composite action between those members, shall be anchored to ~~the supporting walls or pilasters~~ those members in accordance with sections ~~5.1.1.2.1~~5.2.2.1 through ~~5.1.1.2.3~~5.2.2.3.

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~~5.1.1~~5.2.2.1 The supported masonry wall or pilasters shall be anchored so as to transfer no forces other than out-of-plane lateral load acting on the supported wall to the supporting wall.

~~5.1.1~~5.2.2.2 The supported wall or pilasters and the supporting wall shall be permitted to share a common footing or other gravity load support at the base of the walls.

~~5.1.1~~5.2.2.3 The joint and the connectors shall be designed and detailed to accommodate the vertical and horizontal deformations of the supporting wall or pilasters.

~~5.1.1.1~~5.2.3 *Design of masonry wall and pilaster intersections for composite action*

~~5.1.1.1.1~~5.2.3.1 Masonry shall be laid in running bond or shall meet the requirements of Section 5.2.3.5(c).

Commented [PJS75]: 21-SM-PC34

~~5.1.1.1.2~~5.2.3.2 Flanges shall be considered effective in resisting applied loads.

~~5.1.1.1.3~~5.2.2.3 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 and the value shown in Table ~~5.1.1.1.3~~5.2.3, based on the state of stress in the flange and whether or not the masonry is reinforced. The effective flange width shall not extend past a movement joint.

Table ~~5.1.1.1.3~~5.2.3: Effective Flange Width

Stress State in Flange	Unreinforced (U) or Reinforced (R) Masonry	Effective Flange Width
Compression	U, R	6 × nominal flange thickness
Tension	U	6 × nominal flange thickness
	R	0.75 × floor-to-floor wall height

~~5.1.1.1.4~~5.2.3.4 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 9.3.3.1.2; or Section 10.8; or Section 11.3.4.1.2.

~~5.1.1.1.5~~5.2.3.5 The connection of intersecting walls and walls to pilasters 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: $^1/_4$ in. x $1^1/_2$ in. x 28 in. (6.4 mm x 38.1 mm x 711 mm) including 2-in. (50.8-mm) long, 90-degree bend at each end to form a U or Z shape.

(2) Maximum spacing: 48 in. (1219 mm).

(c) Intersecting reinforced bond beams shall be provided at a maximum spacing of 48 in. (1219 mm) on center. The area of reinforcement in each bond beam shall not be less than 0.1 in.^{2} per ft (211 mm^2/m) multiplied by the vertical spacing of the bond beams in feet (meters). Reinforcement shall be developed on each side of the intersection.

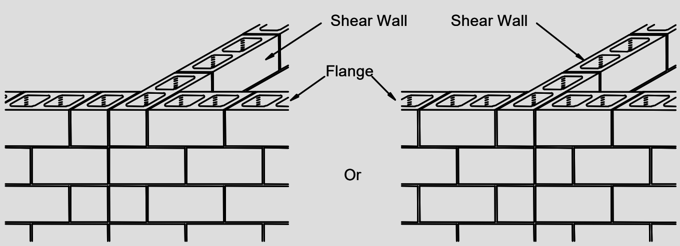

Figure CC-~~5.1-1~~5.2-1 — Running bond lap at intersection

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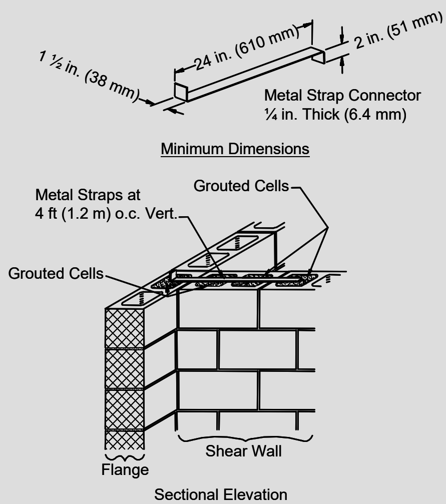
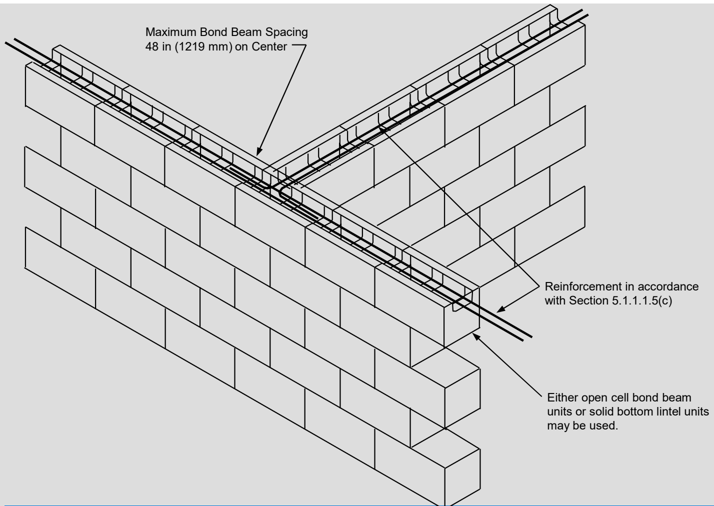


Figure CC-~~5.1-2~~5.2-2 — Metal straps and grouting at wall intersections



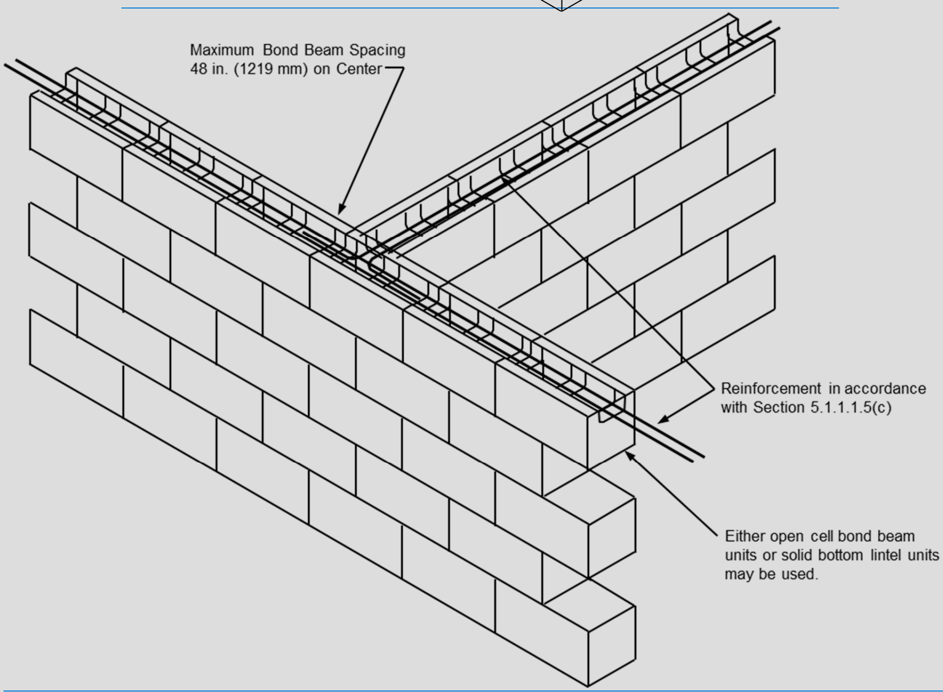


Figure CC-~~5.1-3~~5.2-3 — Bond beam at wall intersection

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~~**5.1.1.2** *Design of lateral supports for walls, without composite action at the intersections*~~

~~Masonry walls depending upon masonry supporting walls or pilasters for lateral support, without composite action between those members, shall be anchored to the supporting walls or pilasters in accordance with sections 5.1.1.2.1 through 5.1.1.2.3.~~

~~**5.1.1.2.1** The supported masonry wall or pilasters shall be anchored so as to transfer no forces other than out-of-plane lateral load acting on the supported wall to the supporting wall.~~

~~**5.1.1.2.2** The supported wall or pilasters and the supporting wall shall be permitted to share a common footing or other gravity load support at the base of the walls.~~

~~**5.1.1.2.3** The joint and the connectors shall be designed and detailed to accommodate the vertical and horizontal deformations of the supporting wall or pilasters .~~

~~**5.1.1.3** *Design of independent walls*~~

~~**5.1.1.3.1** Wall intersections shall be designed and detailed such that no forces are transferred between walls.~~

~~**5.1.2** *Effective compressive width*~~

~~**5.1.2.1** For masonry not laid in running bond and having bond beams spaced not more than 48 in. (1219 mm) center-to-center, and for masonry laid in running bond, the width of the compression area used to calculate member capacity shall not exceed the least of:~~

~~(a) Center-to-center spacing of the reinforcement.~~

~~(b) Six multiplied by the nominal wall thickness.~~

~~(c) 72 in. (1829 mm).~~

~~**5.1.2.2** For masonry not laid in running bond and having bond beams spaced more than 48 in. (1219 mm) center-to-center, the width of the compression area used to calculate member capacity shall not exceed the length of the masonry unit.~~

COMMENTARY

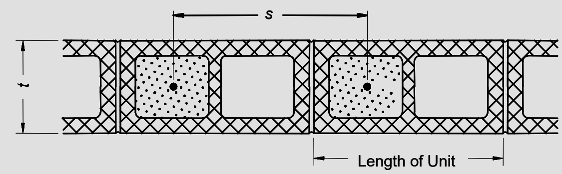
~~**5.1.2** *Effective compressive width*~~

~~The effective width of the compressive area for each reinforcing bar or deformed wire must be established. Figure CC-5.1-4 depicts the limits for the conditions stated. Limited research (Dickey and MacIntosh (1971)) is available on this subject.~~

~~The limited ability of head joints to transfer stress when masonry is not laid in running bond is recognized by the requirements for bond beams. Open end masonry units that are fully grouted are assumed to transfer stress across the head joints even when not laid in running bond.~~

~~The center-to-center reinforcement spacing maximum is a limit to keep from overlapping areas of compressive stress. The 72-in. (1829-mm) maximum is an empirical choice of the Committee.~~

~~COMMENTARY~~



~~For masonry not laid in running bond with bond beams spaced less than or equal to 48 in. (1219 mm) center-to-center and running bond masonry, *b* equals the lesser of:~~

~~$b = s$~~

~~$b = 6t$~~

~~b = 72 in. (1829 mm)~~

~~For masonry not laid in running bond with bond beams spaced greater than 48 in. (1219 mm) center-to-center, *b* equals the lesser of:~~

~~$b = s$~~

~~b = length of unit~~

~~*Figure CC-5.1-4 — Width of compression area*~~

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~~**5.1.4** *Multiwythe masonry*~~

~~Design of masonry composed of more than one wythe shall comply with the provisions of Section 5.1.4.1, and either 5.1.4.2 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, masonry veneer, and glass masonry units.~~

~~**5.1.4.2** *Composite action*~~

~~**5.1.4.2.1** Multiwythe masonry designed for composite action shall have wythes connected by either:~~

~~(a) masonry headers, or~~

~~(b) collar joints and 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 3 in. (76.2 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 wall ties according to Table 5.1.4.2.3.~~

~~COMMENTARY~~

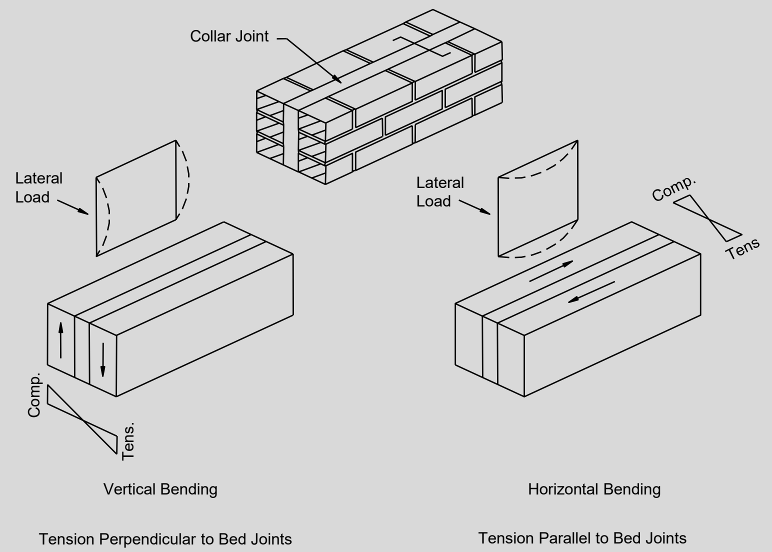
~~**5.1.4.2** *Composite action* Multiwythe masonry acts monolithically if sufficient shear transfer can occur across the interface between the wythes. See Figure CC-5.1-6. Shear transfer is achieved with headers that connect the wythes or with collar joints. When collar joints are relied upon to transfer shear, wall ties are required to ensure structural integrity of the collar joint. Composite action requires that the shear stresses occurring at the interfaces are within the limits prescribed.~~

~~Composite masonry walls generally consist of brick-to-brick, block-to-block, or brick-to-block wythes. The collar joint thickness ranges from $^3/_8$ to 4 in. (9.5 to 102 mm). The joint may contain either vertical or horizontal reinforcement, or reinforcement may be placed in either the brick or block wythe. Composite masonry is particularly advantageous for resisting high loads, both in-plane and out-of-plane.~~

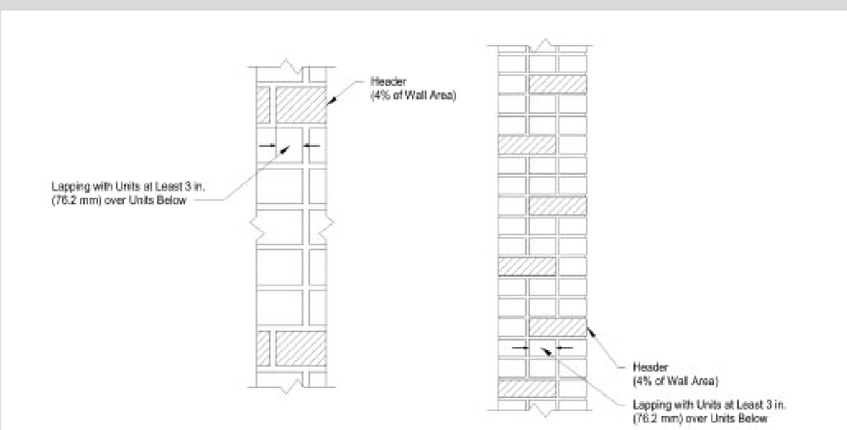
~~**5.1.4.2.2** Requirements for masonry headers (Figure CC-5.1-7) are empirical and taken from prior codes. The net area of the header should be used in calculating the stress even if a solid unit, which is allowed to have up to 25 percent coring, is used. Headers are less ductile than metal wall ties, making differential movement a critical issue, particularly when headers are used to bond two dissimilar masonry materials together such as clay masonry and concrete masonry. Differential movement can shear the headers, eliminating composite action.~~

~~**5.1.4.2.3** The required size, number, and spacing of wall ties in composite masonry, shown in Figure CC-5.1-8, has been determined from past experience. The limitation of Z-ties to masonry of other than hollow units is also based on past experience.~~

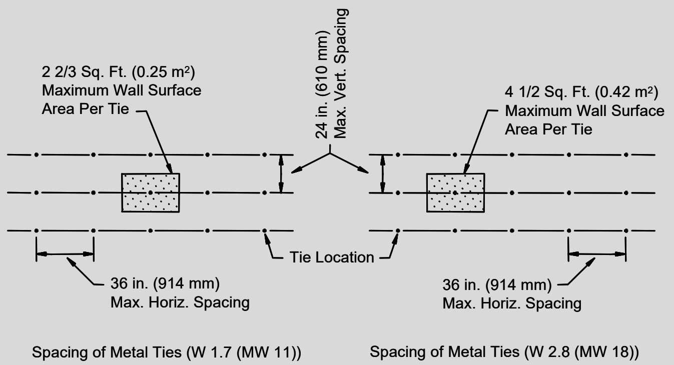
~~COMMENTARY~~



~~Figure CC-5.1-6 — Stress distribution in composite masonry~~



~~Figure CC-5.1-7 — Cross section of wall elevations~~



~~*Figure CC-5.1-8 — Required wall tie spacing for composite masonry*~~

~~Table 5.1.4.2.3: Maximum Spacing and Wall Area for Wall Ties in Multiwythe Masonry Designed per Chapter 8, Chapter 9, or Chapter 10~~

~~Masonry Design Approach~~	**~~Tie Type and Size (cavity drips not permitted)~~**			
	~~Adjustable Wire[1]~~	**~~Non-Adjustable Wire~~**		**~~Joint Reinforcement~~**
	~~W2.8 (MW18)~~	~~W1.7 (MW11)~~	~~W2.8 (MW18)~~	~~W1.7 (MW11) minimum wire~~
~~Masonry Designed for Non-Composite Action~~				
~~Maximum Wall Area per Tie~~	~~1.77 ft² (0.16 m²)~~	~~2.67 ft² (0.25 m²)~~	~~4.50 ft² (0.42 m²)~~	~~Same as non-adjustable unit ties of same wire size~~
~~Maximum Horizontal Spacing~~	~~16 in. (406 mm)~~	~~36 in. (914 mm)~~	~~36 in. (914 mm)~~	~~16 in. (406 mm)~~
~~Maximum Vertical Spacing~~	~~16 in. (406 mm)~~	~~24 in. (610 mm)~~	~~24 in. (610 mm)~~	~~24 in. (610 mm)~~
~~Masonry Designed for Composite Action~~				
~~Maximum Wall Area per Tie~~	~~Not permitted~~	~~2.67 ft² (0.25 m²)~~	~~4.50 ft² (0.42 m²)~~	~~Same as non-adjustable unit ties of same wire size~~
~~Maximum Horizontal Spacing~~	~~Not permitted~~	~~36 in. (914 mm)~~	~~36 in. (914 mm)~~	~~16 in. (406 mm)~~
~~Maximum Vertical Spacing~~	~~Not permitted~~	~~24 in. (610 mm)~~	~~24 in. (610 mm)~~	~~24 in. (610 mm)~~

~~[1] A minimum of two pintle legs and two horizontal legs are required.~~

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~~**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) Cavities 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 shall be distributed to each wythe in proportion to its relative stiffness.~~

~~(c) Lateral loads acting parallel to the plane of the masonry 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 shall be resisted by all wythes in proportion to their relative flexural stiffnesses.~~

~~(e) Specified distances between wythes shall not exceed 4.5 in. (114 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 wall ties according to Table 5.1.4.2.3. Where the cross wires of joint reinforcement are used as wall ties, the joint reinforcement shall be ladder-type or tab-type and shall conform with spacing requirements of Table 5.1.4.2.3. Wall ties shall be without cavity drips.~~

~~COMMENTARY~~

~~**5.1.4.3** *Non-composite action* — Multiwythe masonry may be constructed so that each wythe is separated from the others by a space that may be crossed only by wall ties. The wall ties force compatible lateral deflection, but no composite action exists in the design.~~

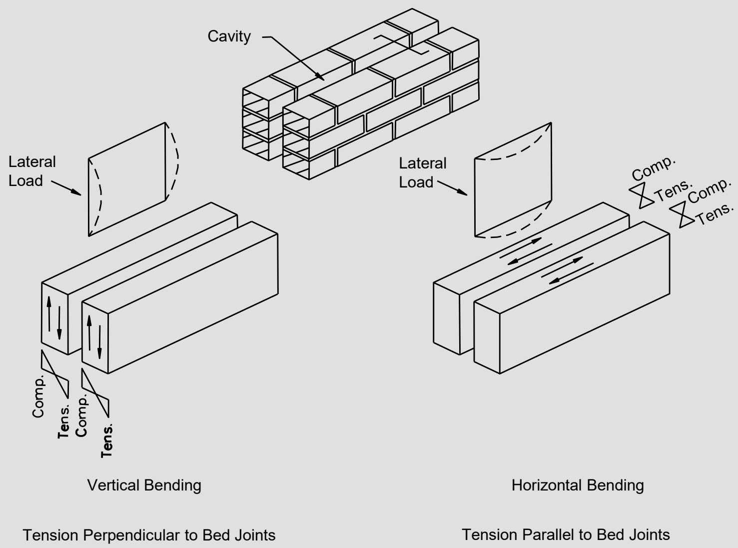
~~**5.1.4.3.1** Weak-axis bending moments caused by either gravity loads or lateral loads are assumed to be distributed to each wythe in proportion to its relative stiffness. See Figure CC-5.1-9 for stress distribution in non-composite masonry. In non-composite masonry, the plane of the masonry is the plane of the space between wythes. Loads due to supported horizontal members are to be resisted by the wythe closest to center of span as a result of the deflection of the horizontal member.~~

~~In non-composite masonry, this Code limits the thickness of the cavity to 4.5 in. (114 mm) to assure adequate performance. If cavity width exceeds 4.5 in. (114 mm), the wall ties must be designed to resist the loads imposed upon them based on a rational analysis that takes into account buckling, tension, pullout, and load distribution.~~

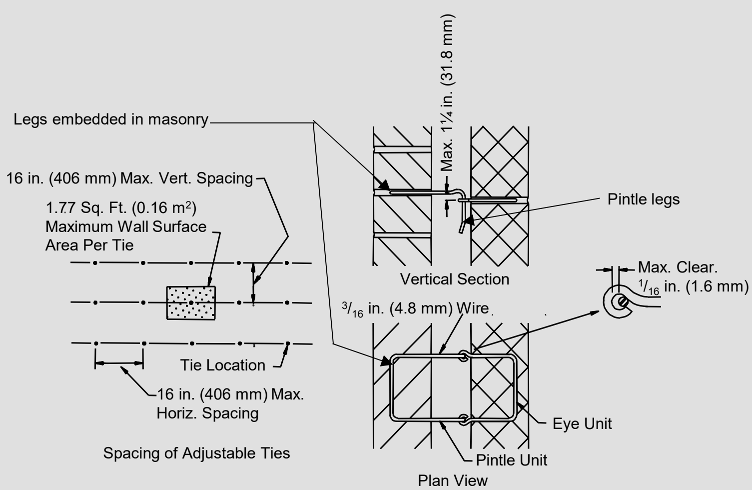
~~The NCMA and Canadian Standards Association (NCMA TEK 12-1B (2011); CSA (2014)) have recommendations for use in the design of ties for masonry with wide cavities.~~

~~**5.1.4.3.2** The required size, number, and spacing of metal wall ties in non-composite masonry (Figure CC-5.1-8) have been determined from past experience. Requirements for adjustable wall ties are shown in Figure CC-5.1-10. They are based on the results in IIT (1963). Ladder-type or tab-type joint reinforcement is required because truss-type joint reinforcement restricts in-plane differential movement between wythes. However, the use of ties with drips (bends in ties to prevent moisture migration) has been eliminated because of their reduced strength.~~

~~COMMENTARY~~



~~Figure CC-5.1-9 — Stress distribution in non-composite multiwythe masonry~~



~~Figure CC-5.1-10 — Required spacing of adjustable wall ties for non-composite masonry~~

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~~5.2~~5.3 — Beams

Design of beams shall meet the requirements of Section ~~5.2~~5.3.1 or Section ~~5.2~~5.3.2. 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~~5.3.1 *General beam design*

~~5.2~~5.3.1.1 *Span length* ~~— Span length shall be in accordance with the following:~~For design of beams other than those designed as deep beams per section 5.2.2, Minimum span length shall be the distance from face-to-face of supports, plus ½ of the required bearing length at each end.

~~**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.~~

~~**5.2.1.1.3** For determination of moments in beams that are built integrally with supports, the span length shall be determined based upon the principles of engineering mechanics, considering the actual end conditions, but shall be permitted to be taken as the distance between centers of supports.~~

~~5.2~~5.3.1.2 *Construction* — Beams requiring transverse shear reinforcement shall be fully grouted.

~~5.2~~5.3.1.3 *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~~5.3.1.4 *Bearing length* — Length of bearing of beams on their supports shall be a minimum of 4 in. (102 mm) in the direction of span.

COMMENTARY

~~5.2~~5.3 — Beams

All masonry beams are required to be reinforced to provide strength and ductility.

~~5.2~~5.3.1 *General beam design*

~~5.2~~5.3.1.1 *Span length*

~~5.2~~5.3.1.2 *Construction* — Prior to the 2022 edition of this Code, beams designed using the strength design provisions of Chapter 9 and Chapter 11 were required to be fully grouted, while beams designed using the allowable stress design provisions of Chapter 8 were not required to be fully grouted. The full grouting requirement of beams designed using the strength design provisions of Chapter 9 and Chapter 11 has been removed and partially grouted masonry beams are permitted. Full grouting of beams is still required when transverse shear reinforcement is required by analysis. From a construction viewpoint, grouting is preferable to shear reinforcement in beams.

~~5.2~~5.3.1.3 *Lateral support* — To minimize lateral torsional buckling, the Code requires lateral bracing of the compression face. Hansell and Winter (1959) suggest that the slenderness ratios should be given in terms of $\ell d/b^2$. Revathi and Menon (2006) report on tests of seven under-reinforced slender concrete beams. In Figure CC-~~5.2~~5.3-1, a straight line is fitted to the W_{test}/W_u ratio vs. $\ell d/b^2$, where W_{test} is the experimental capacity and W_u is the calculated capacity based on the full cross-sectional moment strength. W_{test}/W_u equals 1 where $\ell d/b^2$ equals 146. Based on this, the Code limit of 120 for $\ell d/b^2$ is reasonable and slightly conservative.

~~5.2~~5.3.1.4 *Bearing length* — The minimum bearing length of 4 in. (102 mm) in the direction of span is considered a reasonable minimum to reduce concentrated compressive stresses at the edge of the support.

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~~5.2~~5.3.1.5 *Shear* — In cantilever beams, the maximum shear shall be used. In noncantilever beams, the maximum shear shall be used except that sections located within a distance $d/2$ from the face of support shall be permitted to 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.

~~5.2~~5.3.1.5 *Shear* — The beam loading within $d/2$ of the support is assumed to be transferred in direct compression or tension to the support without increasing the shear force, provided no concentrated load occurs within the $d/2$ distance.

~~5.2~~5.3.1.6 *Deflections* — Masonry beams shall be designed to have adequate stiffness to limit deflections that adversely affect strength or serviceability.

~~5.2~~5.3.1.6 *Deflections* — The provisions of Section ~~5.2~~5.3.1.6 address deflections that may occur at allowable stress level loads.

~~5.2~~5.3.1.6.1 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~~5.3.1.6.1 ~~Reinforced m~~Masonry beams and lintels with span lengths of 8 times d have immediate deflections of approximately 1/600 of the span length (Bennett et al (2007)). Masonry beams and lintels with shorter spans should have sufficient stiffness to prevent serviceability problems and, therefore, deflections do not need to be checked.

Given that no other deflection limit is currently provided in the Code, reinforced masonry beams and lintels (with spans longer than 8 times d) supporting reinforced masonry may be conservatively checked against the deflection limits stated in Section ~~4.5~~4.6 and its Commentary. When further research data specific to masonry beams becomes available, a less stringent deflection limit (than l/600) may be shown to be applicable to reinforced masonry beams and lintels supporting reinforced masonry. Alternatively, the engineer may use the deflection limits listed in the IBC for roof and floor members, as applicable.

~~5.2~~5.3.1.6.2 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.

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

(Equation 5-1)

For continuous beams, I_{eff} shall be permitted to be taken as the average of values obtained from Equation 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 Equation 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

~~5.2~~5.3.1.6.2 The effective moment of inertia was developed to provide a transition between the upper and lower bounds of I_g and I_{cr} as a function of the ratio M_{cr}/M_a (Branson (1965)). This procedure was selected as being sufficiently accurate for use to control deflections (Horton and Tadros (1990)). Calculating a more accurate effective moment of inertia using a moment-curvature analysis may be desirable for some circumstances.

Most masonry beams have some end restraint due to being built integrally with a wall. Tests have shown that the end restraint from beams being built integrally with walls reduces the deflections from 20 to 45 percent of those of the simply supported specimens (Lee et al (1983)).

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using the allowable flexural tensile stress taken from Table 8.2.4.2 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 9.1.9.~~2~~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~~2.

COMMENTARY

COMMENTARY

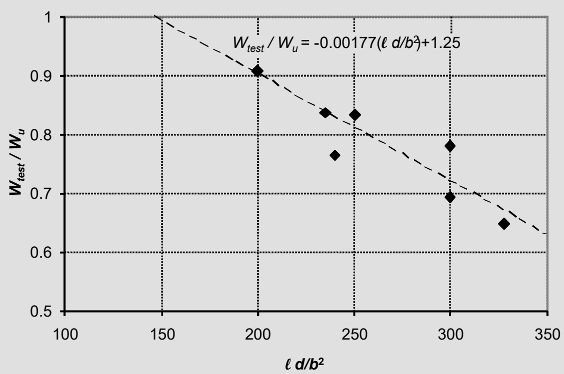

Figure CC-~~5.2~~5.3-1 Beam capacity vs. beam slenderness (after Revathi and Menon (2006))

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~~5.2~~5.3.2 *Deep beams*

Design of deep beams shall meet the requirements of Section ~~5.2~~5.3.1.2, ~~5.2~~5.3.1.3, and ~~5.2~~5.3.1.4 in addition to the requirements of Section ~~5.2~~5.3.2.1 through ~~5.2~~5.3.2.5.

~~5.2~~5.3.2.1 *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.

COMMENTARY

~~5.2~~5.3.2 *Deep beams*

Shear warping of the deep beam cross section and a combination of diagonal tension stress and flexural tension stress in the body of the deep beam require that these members be designed using deep beam theory when the span-to-depth ratio is within the limits given in the definition of deep beams. Background on the development of the deep beam provisions is given in Fonseca et al (2011).

As per the definition in Section 2.2, a deep beam 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. Sections of masonry over openings may be designed as deep beams if the span-to-depth ratio meets these limits. However, the depth of the beam need not be taken as the entire height of masonry above the opening. A shallower beam can be designed to support the remaining portion of the masonry above in addition to applied loads. This beam can be designed conventionally and need not meet the deep beam provisions if sufficiently shallow. (see Figure CC-~~5.2~~5.3-2)

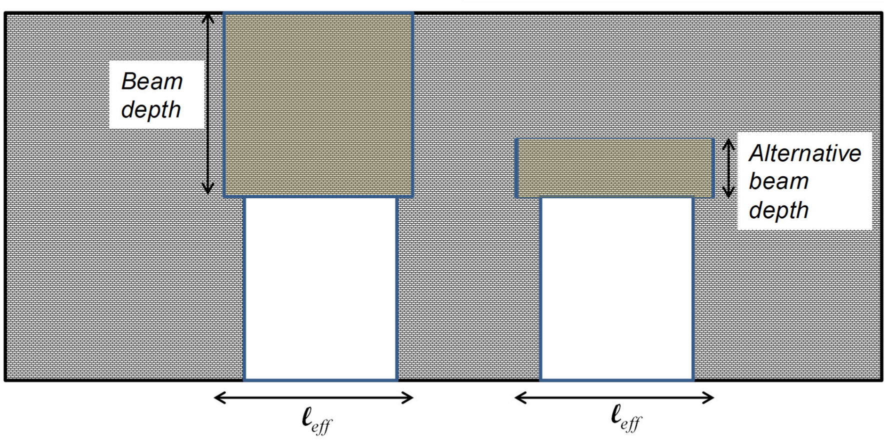

Figure CC-~~5.2~~5.3-2 Possible depth of beams over openings in masonry walls

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~~5.2~~5.3.2.2 *Internal lever arm* — Unless determined by a more comprehensive analysis, the internal lever arm, z, shall be taken as the value in Table ~~5.2~~5.3.2.2, based on the span condition and ratio of ℓ_{eff} / d_v.

~~5.2~~5.3.2.3 *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 16 in. (406 mm). Joint reinforcement shall be permitted to be used as distributed horizontal flexural reinforcement in deep beams. Horizontal flexural reinforcement shall be developed at the face of the support in accordance with Section 6.1.6.

~~5.2~~5.3.2.4 *Minimum shear reinforcement* — The following provisions shall apply when shear reinforcement is required.

(a) The minimum area of vertical shear reinforcement shall be 0.0007 bd_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 8 in. (203 mm).

(c) The maximum spacing of shear reinforcement shall not exceed one-fifth the beam depth, d_v, nor 16 in. (406 mm).

~~5.2~~5.3.2.5 *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.

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~~5.2~~5.3.2.2 *Internal lever arm* — The theory used for design of beams has limited applicability to deep beams. Specifically, there will be a nonlinear distribution of strain in deep beams. The internal lever arm, z, between the centroid of the internal compressive forces and the internal tensile forces will be less than that calculated assuming a linear strain distribution. The Code equations for internal lever arm, z, can be used with either allowable stress design or strength design. For allowable stress design, z is commonly known as jd, and for strength design, z is commonly known as d-$(a/2)$. The internal lever arm provisions in the Code are based on CEB-FIP (~~1990~~2010).

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~~5.2~~5.3.2.3 *Flexural reinforcement* — The distribution of tensile stress in a deep beam is generally such that the lower one-half of the beam is required to have distributed flexural reinforcement. However, other loading conditions, such as uplift, and support conditions, such as continuous and fixed ends, should be considered in determining the portion of the deep beam that is subjected to tension. Distributed horizontal reinforcement resists tensile stress caused by shear as well as by flexure.

~~5.2~~5.3.2.4 *Minimum shear reinforcement* — Distributed flexural reinforcement may be included as part of the provided shear reinforcement to meet the minimum distributed shear reinforcement ratio. The spacing of shear reinforcement is limited to restrain the width of the cracks. Load applied along the top surface of a deep beam is transferred to supports mainly by arch action. Typically, deep beams do not need transverse reinforcement and it is sufficient to provide distributed flexural reinforcement (Park and Paulay (1975)).

Table ~~5.2~~5.3.2.2: Internal Lever Arm

Span Condition	Ratio	Internal Lever Arm (z)
Simply Supported	$1 \le \ell_{eff} / d_v < 2$	$0.2(\ell_{eff} + 2d_v)$
	$\ell_{eff} / d_v < 1$	$0.6\ell_{eff}$
Continuous	$1 \le \ell_{eff} / d_v < 3$	$0.2(\ell_{eff} + 1.5d_v)$
	$\ell_{eff} / d_v < 1$	$0.5\ell_{eff}$

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~~5.3~~5.4 — Columns

Design of columns shall meet the requirements of Section ~~5.3~~5.4.1 or Section ~~5.3~~5.4.2. Design of columns shall also meet the requirements of Section 8.3, or Section 9.3, or Section 11.3.

~~5.3~~5.4.1 *General column design*

~~5.3~~5.4.1.1 *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 8 in. (203 mm) nominal.

~~5.3~~5.4.1.2 *Construction* — Columns shall be fully grouted.

~~5.3~~5.4.1.3 *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.

COMMENTARY

~~5.3~~5.4 — Columns

Columns are defined in Section 2.2. They are isolated members usually under axial compressive loads and flexure. If damaged, columns may cause the collapse of other members; sometimes of an entire structure. These critical structural members warrant the special requirements of this section.

~~5.3~~5.4.1 *General column design*

~~5.3~~5.4.1.1 *Dimensional limits* — The limit of 99 for the slenderness ratio, *h/r*, is judgment based. See Figure CC-~~5.3~~5.4-1 for effective height determination. The minimum nominal side dimension of 8 in. (203 mm) results from practical considerations.

~~5.3~~5.4.1.3 *Vertical reinforcement* — Minimum vertical reinforcement is required in masonry columns to prevent brittle failure. The maximum percentage limit in column vertical reinforcement was established based on the Committee's experience. Four bars are required so lateral ties can be used to provide a confined core of masonry.

If data (see Section 1.3) show that there is reliable restraint against translation and rotation at the supports, the "effective height" may be taken as low as the distance between points of inflection for the loading case under consideration.

Figure CC-~~5.3~~5.4-1 — Effective height, h, of column, wall, or pilaster

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~~5.3~~5.4.1.4 *Lateral ties* — Lateral ties shall conform to the following:

(a) Vertical reinforcement shall be enclosed by lateral ties at least $^{1}/_{4}$ in. (6.4 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 6 in. (152 mm) clear on each side along the lateral tie from such a laterally supported bar. 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 embedded in grout. When a lateral tie or combination of ties does not exceed the specified thickness of the mortar joint, the portion of the tie(s) that crosses a web or interior face shell shall be permitted to be embedded in mortar.

(e) 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~~5.4.2 *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 allowable stress level gravity loads not exceeding 2,000 lbs (8,900 N) or 50 psi (345 kPa) acting within the cross-sectional dimensions of the column are permitted to be constructed as follows:

(a) Minimum side dimension shall be 8 in. (203 mm) nominal.

(b) Height shall not exceed 12 ft (3.66 m).

(c) Cross-sectional area of longitudinal reinforcement shall not be less than 0.2 in.^{2} (129 mm^2) centered in the column.

(d) Columns shall be fully grouted.

These provisions do not apply to AAC masonry columns.

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~~5.3~~5.4.1.4 *Lateral ties* — Lateral reinforcement in columns performs two functions. It provides the required support to prevent buckling of longitudinal column reinforcing bars acting in compression and provides resistance to diagonal tension for columns acting in shear (Pfister, 1964).

The requirements of this Code are modeled on those for reinforced concrete columns. Except for permitting ¼-in. (6.4-mm) lateral ties in Seismic Design Category A, B, and C, they reflect the applicable provisions of the reinforced concrete code.

A corner of a lateral tie provides rigidity to the longitudinal bars being laterally supported. The corner of a lateral tie is formed by a bend, which creates an interior angle that is not permitted to be more than 135 degrees. The limit of 135 degrees does not apply to the hooks at the ends of the lateral tie. When a lateral tie or combination of ties exceeds the specified mortar joint thickness, removal of part of the unexposed portion of the unit(s) or other modification is required to maintain proper clearance and grout coverage.

Figures CC-~~5.3~~5.4-2 and CC-~~5.3~~5.4-3 show examples of an interior angle for a CMU column and for a clay masonry column, respectively.

~~5.3~~5.4.2 *Lightly loaded columns*
Masonry columns are often used to support roofs of carports, porches, sheds or similar light structures. These columns do not need to meet the detailing requirements of Section ~~5.3~~5.4.1. The axial load limit of 2,000 pounds (8,900 N) or 50 psi (345 kPa) was developed based on the flexural strength of a nominal 8 in. (203 mm) by 8 in. (203 mm) by 12 ft high (3.66 m) column with one No. 4 (M#13) reinforcing bar in the center and f'_m of 1350 psi (9.31 MPa). An axial load of 2,000 pounds (8,900 N) at the edge of the member will result in a moment that is approximately equal to the nominal flexural strength of this member. Although the allowable pressure limit may result in a total load of up to 3000 lb (13,340 N), the nature of uniform pressure means that the resultant occurs at the centroid of the pressure. Therefore, the resulting moment will be minor and the total stress on the column (including bending and axial stress) will be less than the total stress from a 2000 lb (8,900 N) load applied at the edge of the section.

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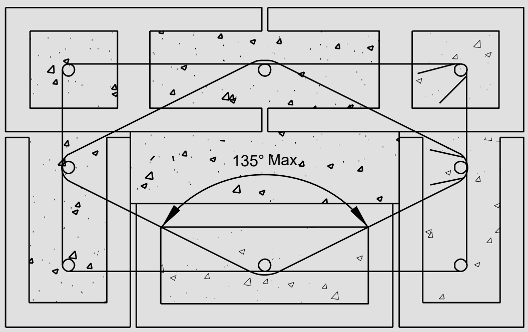

Figure CC-~~5.3~~5.4-2 — Example of a lateral tie included angle for a CMU column

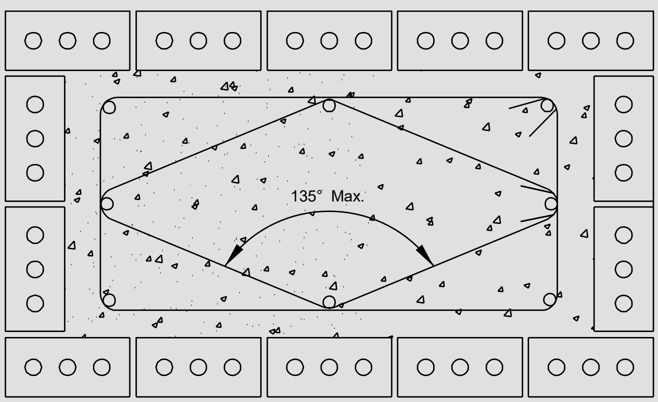

Figure CC-~~5.3~~5.4-3 — Example of a lateral tie included angle for a clay masonry column

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~~5.4~~5.5 — Pilasters

~~5.4~~5.5.1 Walls interfacing with projecting pilasters shall not be considered as flanges, unless placed in running bond andthe construction requirements of Sections ~~5.1.1.1.1 and~~ ~~5.1.1.1.5~~5.2.3.5 are met. When these construction requirements are met, the projecting pilaster's flanges shall be designed in accordance with Sections ~~5.1.1.1.2~~5.2.3.2 through ~~5.1.1.1.4~~5.2.3.4.

~~5.4~~5.5.2 Non-projecting pilasters shall be designed in accordance with the reinforced wall provisions of this Code.

~~5.5~~5.6 — Corbels

~~5.5~~5.6.1 *Load-bearing corbels* Load-bearing corbels shall be designed in accordance with Chapter 8, 9 or 10.

~~5.5~~5.6.2 *Non-load-bearing corbels* Non-load-bearing corbels shall be designed in accordance with Chapter 8, 9 or 10 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 nominal wall thickness for composite multiwythe walls, or
- (2) one-half the nominal wythe thickness for single wythe walls, multiwythe walls with cavities 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 1 in. (25.4 mm) of plane. ACC Masonry shall be designed according to Section 11.1.10.

COMMENTARY

~~5.4~~5.5 — Pilasters

Pilasters are masonry members that can serve several purposes. They may project from one or both sides of the wall, as shown in Figure CC-~~5.4~~5.5-1. Pilasters contribute to the lateral load resistance of masonry walls and may resist vertical loads.

~~5.5~~5.6 — Corbels

The provision for corbelling up to one-half of the nominal wall or wythe thickness is theoretically valid only if the opposite side of the wall remains in its same plane. The addition of the 1-in. (25.4-mm) intrusion into the plane recognizes the impracticality of keeping the back surface plane. See Figure CC-~~5.5~~5.6-1 and CC-~~5.5~~5.6-2 for maximum permissible unit projection.

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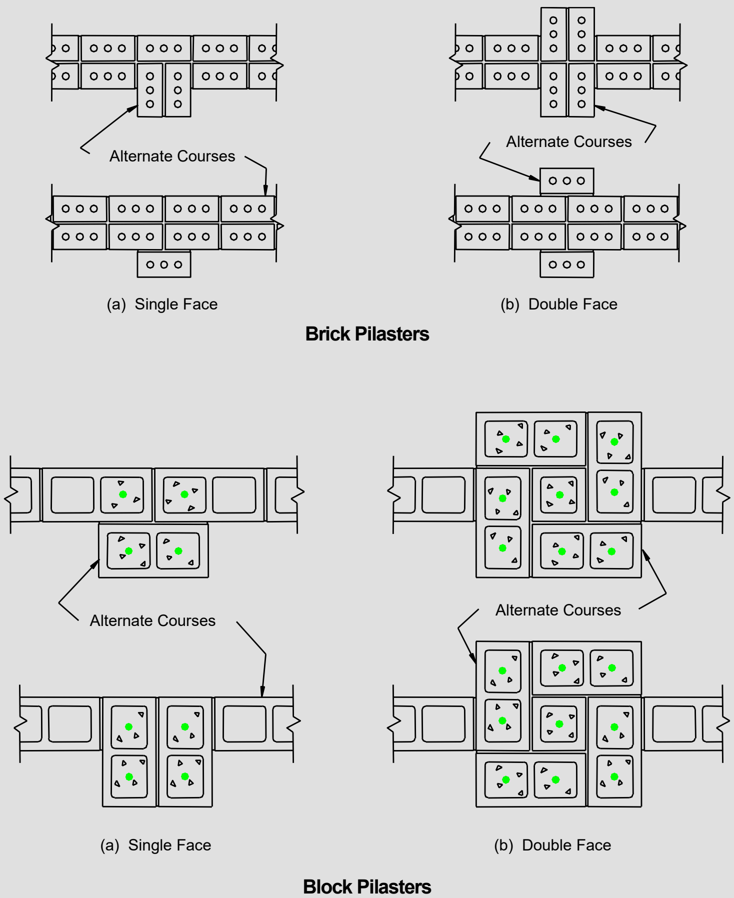

Figure CC-~~5.4~~5.5-1 — Typical pilasters (Lateral ties are not shown)

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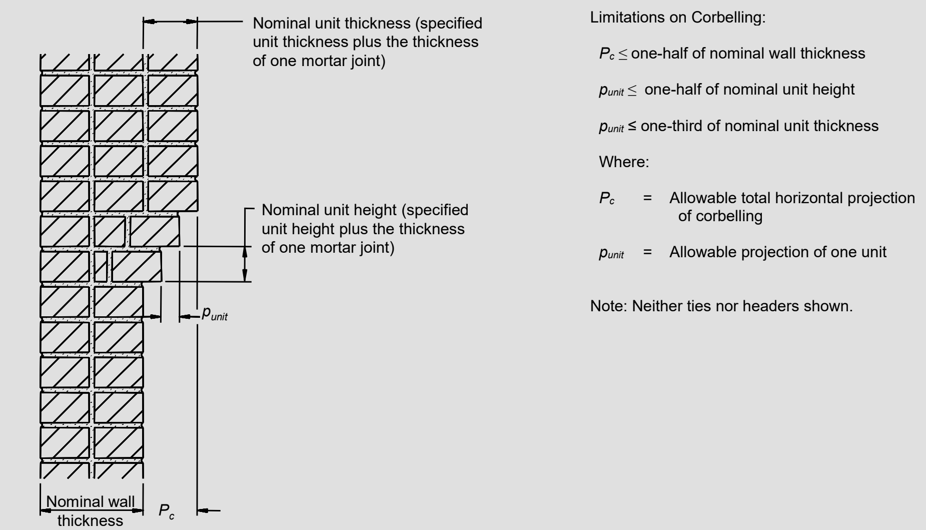

Figure CC-~~5.5~~5.6-1 — Limits on corbelling in composite multiwythe walls

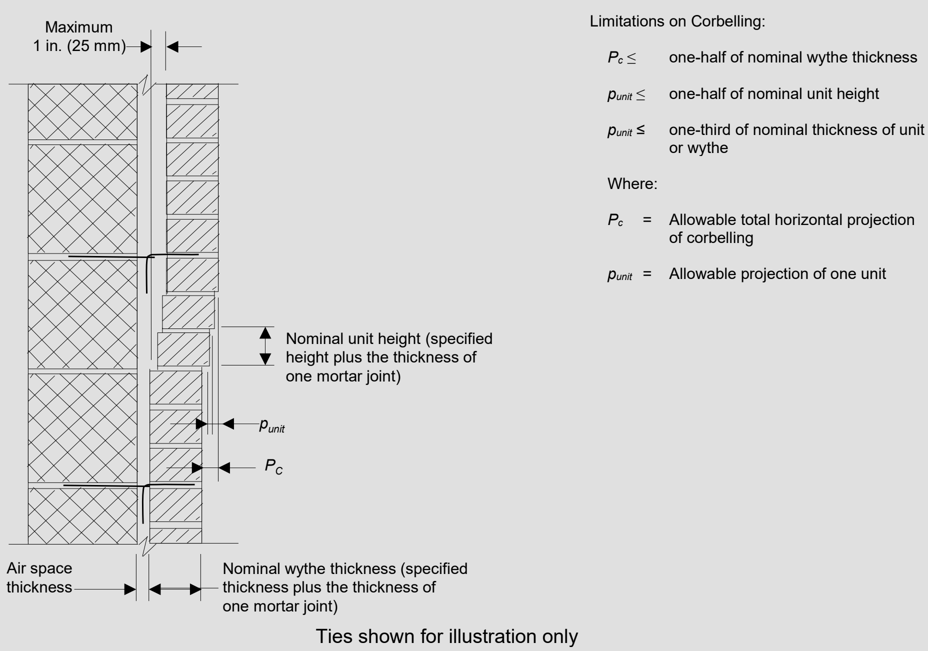

Figure CC-~~5.5~~5.6-2 – Limits on corbelling in walls with air space

CHAPTER 6
REINFORCEMENT, METAL ACCESSORIES, AND ANCHOR BOLTS

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6.1 — Reinforcement

6.1.1 *Scope*

The provisions of Section 6.1 shall apply to reinforcement that consists of one or more of the following:

(a) deformed reinforcing bars that conform to TMS 602 Article 2.4 A; or

(b) joint reinforcement that conforms to TMS 602 Article 2.4 D; or

(c) deformed reinforcing wire that conforms to TMS 602 Article 2.4 ~~E~~F and having a specified yield strength not greater than 75 ksi (515 MPa); or

(d) welded deformed wire reinforcement that conforms to TMS 602 Article 2.4 ~~F~~G and having a specified yield strength not greater than 70 ksi (485 MPa).

6.1.2 *Embedment*

Reinforcing bars and welded wire reinforcement shall be embedded in grout except as permitted by Section ~~5.3~~5.4.1.4 (d).

6.1.3 *Size of reinforcement*

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6.1 — Reinforcement

When the provisions of this section were originally developed in the late 1980s, the Committee used the 1983 edition of the ACI 318 Code as a guide. Some of the requirements were simplified and others dropped, depending on their suitability for application to masonry.

6.1.1 *Scope*

(a) Deformed reinforcing bars can be placed in grouted vertical cells, bond beams, grouted collar joints and other similar grouted spaces.

(b) Joint reinforcement is placed in mortared bed joints.

(c) Deformed reinforcing wire may be placed either in mortared joints or in grout as described for deformed reinforcing bars. The limitation on yield strength for deformed wires was adopted in the 2022 edition based on the strength of materials that were used in the testing that formed the basis of the development and lap splice length provisions of this Chapter.

(d) Although not often used in masonry construction, welded wire reinforcement provides a convenient means of placing reinforcement in a grouted collar joint. The limitation on the yield strength for welded deformed wire reinforcement has the same basis as the limit on deformed wire.

6.1.2 *Embedment*

Welded wire reinforcement is required to be placed in grout to accommodate the intersecting wires and for consistency with the development and splice provisions.

6.1.3 *Size of reinforcement*

Properties of bar and wire reinforcement are given in Table CC-6.1.3. The listed data for wire, which are based on ASTM A1064, provide calculated equivalent designations in SI units for reference although wires in those sizes may not be produced as metric wires. Not all incremental wire size data from ASTM A1064 are included to keep the table size reasonable while providing data for wires most useful to designs based on this Code. The Code requires single wire reinforcement to be deformed per Section 6.1.1.c, as does Specification Article 2.4~~E~~F. W1.7 (9 gage) and W2.8 (3/16 inch) plain wires are not listed in ASTM A1064 but are included in Table CC- 6.1.3 because these wire sizes are used to produce joint reinforcement. The industry commonly uses the term “standard joint reinforcement” to refer to joint reinforcement with W1.7 (9 gage) longitudinal wires and the term “heavy-duty joint reinforcement” to refer to joint reinforcement with W2.8 (3/16 inch) longitudinal wires. Although joint

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reinforcement longitudinal wires are knurled, they are not deformed in accordance with ASTM A1064 and, therefore, are designated as plain wires.

Limitations on the size of mechanical splices are provided in Section 6.1.7.2.3.

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6.1.3.1 *Size of reinforcement in mortar*

6.1.3.1.1 The maximum size of deformed wire placed in a mortar joint shall be one-half the mortar joint thickness. The minimum diameter of deformed wire used in masonry shall be 0.10 in. (2.5 mm).

6.1.3.1.2 Longitudinal and cross wires of joint reinforcement shall have a maximum wire size of one-half the joint thickness and a minimum wire size of W1.7 (MW11).

6.1.3.2 *Size of reinforcement in grout*

6.1.3.2.1 The maximum size of bar reinforcement used in masonry shall be No. 11 (M #36).

6.1.3.2.2 The maximum size of deformed wire placed in grout shall be D31 (MD 200).

6.1.3.1 *Size of reinforcement in mortar*

6.1.3.1.1 The restriction on wire size ensures adequate performance. The maximum wire size of one-half the joint thickness allows free flow of mortar around joint reinforcement. Thus, a 3/16-in. (4.8- mm) diameter wire can be placed in a 3/8-in. (9.5-mm) joint.

6.1.3.1.2 The function of joint reinforcement is to control the size and spacing of cracks caused by volume changes in masonry as well as to resist tension (Dickey (1982)). Joint reinforcement is commonly used in concrete masonry to minimize shrinkage cracking. Refer to Commentary 6.1.3.1.1 for additional background on wire size restrictions.

When joint reinforcement with 3/16-in. (4.8-mm) diameter longitudinal wires are specified to be placed in every course (as required by Sections 7.4.1.2.1 and 7.4.3.2.6 for masonry in SDC C and higher), the permitted construction tolerances should be adjusted. Larger tolerances are needed because the joints may need to be oversized to accommodate the larger joint reinforcement, taking advantage of the ±1/8 in. (3.2 mm) tolerance in bed joint thickness permitted by TMS 602. When the joint reinforcement is placed at every other course, the oversized joints that accommodate the joint reinforcement can be compensated for by undersizing the joints without joint reinforcement. This allows the wall and openings within the wall to be constructed to the specified elevation and size. When every joint is occupied by joint reinforcement with 3/16-in. (4.8-mm) diameter longitudinal wires, there is no opportunity to undersize mortar joints and it may not be possible to construct the wall to the specified geometry.

Joint reinforcement is available in ladder type (perpendicular cross wires) and truss type (diagonal cross wires). Where vertical reinforcement is present in a masonry wall, diagonal wires in truss type joint reinforcement will conflict with placement of the vertical reinforcement. Mortar droppings on the diagonal cross wires also make quality grouting more difficult. Consequently, truss-type joint reinforcement should not be specified when the masonry contains vertical reinforcement.

6.1.3.2 *Size of reinforcement in grout*

6.1.3.2.1 The No. 11 (M#36) limit is arbitrary, but Priestley and Bridgeman (1974) and Noland and Kingsley (1995) show that distributed small bars provide better performance than fewer large bars.

6.1.3.2.2 The upper limit on size of deformed wire is based on testing of deformed wire and bar reinforcement in concrete by Rutledge and DeVries (2002), which demonstrated that D45 (MD 290) deformed wire was

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6.1.3.2.3 The nominal bar or wire diameter shall not exceed one-eighth of the least nominal member dimension.

6.1.3.2.4 The diameter of reinforcement shall not exceed one-third the least dimension of the gross grout space in which it is placed.

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unable to develop the same bond strength in tension as the corresponding size of reinforcing bar.

6.1.3.2.3 This requirement is based on the research conducted by Soric and Tulin (1987).

6.1.3.2.4 Adequate flow of grout necessary for good bond is achieved with this limitation.

Table CC-6.1.3: Physical properties of steel bar and wire reinforcement

Designation	Nominal Diameter, in. (mm)	Nominal Area, in.^{2} (mm^2)
Deformed Bars		
No. 3 (M#10)	0.375 (9.5)	0.11 (71.0)
No. 4 (M#13)	0.500 (12.7)	0.20 (129)
No. 5 (M#16)	0.625 (15.9)	0.31 (199)
No. 6 (M#19)	0.750 (19.1)	0.44 (284)
No. 7 (M#22)	0.875 (22.2)	0.60 (387)
No. 8 (M#25)	1.000 (25.4)	0.79 (510)
No. 9 (M#29)	1.128 (28.7)	1.00 (645)
No. 10 (M#32)	1.270 (32.3)	1.27 (819)
No. 11 (M#36)	1.410 (35.8)	1.56 (1006)
Deformed Wire		
D 1 (MD 6.5)	0.113 (2.87)	0.010 (6.47)
D 2 (MD 13)	0.160 (4.06)	0.020 (13.0)
D 3 (MD 19)	0.195 (4.95)	0.030 (19.3)
D 4 (MD 26)	0.226 (5.74)	0.040 (25.9)
D 5 (MD 32)	0.252 (6.40)	0.050 (32.2)
D 6 (MD 39)	0.276 (7.01)	0.060 (38.6)
D 7 (MD 45)	0.299 (7.59)	0.070 (45.3)
D 11 (MD 71)	0.374 (9.50)	0.110 (70.9)
D 20 (MD 129)	0.505 (12.8)	0.200 (129)
D 31 (MD 200)	0.628 (16.0)	0.310 (200)
Plain Wire		
W1.7 (MW 11) (9 gage)	0.148 (3.8)	0.017 (11.0)
W 2 (MW 13)	0.160 (4.05)	0.020 (12.9)
W 2.5 (MW 16)	0.178 (4.53)	0.025 (16.1)
W2.8 (MW 17) (3/16 in. wire)	0.187 (4.8)	0.027 (17.4)
W 2.9 (MW 19)	0.192 (4.88)	0.029 (18.7)
W 4 (MW 26)	0.226 (5.73)	0.040 (25.8)
W4.9 (MW 32) (1/4 in. wire)	0.250 (6.4)	0.049 (31.6)

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6.1.3.2.5 The area of vertical reinforcement and area of horizontal reinforcement shall each not exceed the percentages of the gross grout space defined by Table 6.1.3.2.5, based on the masonry material, location of the lap splice, and whether lapped reinforcement is included in the percentage.

Table 6.1.3.2.5: Maximum Reinforcement Percentages of Gross Grout Space

Masonry Material / Splice Location	Laps Excluded	Laps Included
Clay masonry and concrete masonry/any splice location	4%	8%
AAC masonry/splices in plastic hinge zones	3%	6%
AAC masonry/splices in other than plastic hinge zones	4.5%	9%

COMMENTARY

6.1.3.2.5 The limitations on maximum reinforcement percentage are based on the gross grout space presented by the cell, bond beam course, grouted collar joint, or AAC masonry core. These limitations are in contrast to the requirements for grout placement in TMS 602 Table 7, which are based on the net grout space per Footnote 3 and TMS 602 Figure SC-21. The limitations of Section 6.1.3.2.5 are intended to avoid overreinforcing, while the limitations of TMS 602 Article 3.5C are intended to prevent problems with grout consolidation.The alternative provisions presented in Table 6.1.3.2.5.1 and Table 6.1.3.2.5.2 provide a simplified method of determining the maximum vertical reinforcement permitted by TMS 402 when designing vertically reinforced two-celled hollow concrete masonry and hollow clay masonry even though the dimensions of the unit cross-section are unknown before the units have been ordered by the contractor. Because these provisions are simplified, they are also conservative. Designers who know the cross-sectional dimensions of the units to be used on the project may be able to specify greater amounts of reinforcement than those shown in these Tables, especially for units greater than 6-in. (152 mm) in thickness. The percentages in these Tables were correlated to the values in Table 6.1.3.2.5 and are based on "per 8-in. (203 mm) length" (per cell or core for two-celled units), with a footnote to address nominal 12-in. (305 mm) long clay units that have a 6-in. (152 mm) length per core or cell. Table 6.1.3.2.5.1 applies to units laid in one-half running bond (units overlap 50% of their length) and Table 6.1.3.2.5.2 applies to units laid in stack bond (unit overlap 100% of their length). Figure CC-6.1-1 illustrates two-celled flanged units, jamb units, and open-end units laid in one-half running bond for typical CMU and clay units.

Concrete and clay masonry unit configurations can vary regionally and between manufacturers due to local production preferences. Consult producers local to the project to develop expected unit geometric parameters prior to calculating gross grout space. Other detailing aspects such as corbeling and varied unit overlap can also affect the available gross grout space. Include sufficient notes and/or details to illustrate necessary unit geometry and unit placement limits for compliance with the design basis. Refer to Figure CC-6.1-1 for illustrations of several unit possibilities in one-half running bond pattern. Other bond patterns and unit alignments may require consideration in the calculation of gross grout space.

Section 6.1.3.2.5, and Table 6.1.3.2.5, have been developed for use with a calculated gross grout space area and that space is the gross area available for grout based solely on the unit geometric properties and placement (bond, alignment, corbeling, etc.). Note that concrete masonry units typically include a taper for mold removal and, therefore, are thicker at their tops and the maximum thickness should be used when calculating the gross grout space. Structural clay units are typically extruded and maintain constant wall thickness throughout their depth.

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6.1.3.2.5.1 Alternatively, vertical reinforcement in two-celled hollow concrete masonry and hollow clay masonry shall not exceed the percentages given in Table 6.1.3.2.5.1, based on the unit shape, the specified gross area of masonry wythe per 8-in. (203 mm) length, and whether lapped bars are included in the percentage.

Table 6.1.3.2.5.1: Maximum Vertical Reinforcement Percentages Based on Specified Wythe Thickness and 8-in (203 mm) Length When Laid in One-Half Running Bond [1,2,3]

Unit Shape	Laps Excluded	Laps Included
Flanged ends	1.1 %	2.2%
Jamb ends	1.5%	3%
Open ends	2%	4%

[1] Two-celled hollow concrete masonry and hollow clay masonry

[2] Limited to hollow concrete units from 6-in. (152 mm) nominal thickness to 12-in. (305 mm) nominal thickness and to hollow clay units from 4-in. (102 mm) nominal thickness to 8-in. (203 mm) nominal thickness.

[3] For clay masonry units with a 6-in. (152 mm) vertical reinforcement module, multiply the tabular value by 0.75 to determine the maximum vertical reinforcement percentage per 6-in. (152 mm) length.

6.1.3.2.5.2 Alternatively, vertical reinforcement in two-celled hollow concrete masonry and hollow clay masonry shall not exceed the percentages given in Table 6.1.3.2.5.2, based on the unit shape, the specified gross area of masonry wythe per 8-in. (203 mm) length, and whether lapped bars are included in the percentage.

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The effects of other items such as mortar extrusions, vertical and horizontal bars, etc., should not be included in the calculation of gross grout space.

Table CC-6.1.3.2.5.1 shows the maximum size and quantity of vertical reinforcement permitted by Sections 6.1.3.2.5, 6.1.3.2.5.1, and 6.1.3.2.2 for two-celled masonry units laid in one-half running bond. Table CC-6.1.3.2.5.2 shows the maximum size and quantity of vertical reinforcement permitted by Sections 6.1.3.2.5, 6.1.3.2.5.2, and 6.1.3.2.2 for two-celled masonry units laid in stack bond. Tables CC-6.1.3.2.5.1 and CC-6.1.3.2.5.2 do not include nominal unit thicknesses less than 6-in. (152 mm) as there are no commercially available two-celled units with an 8-in. (203 mm) module. Table CC-6.1.3.2.5.3 shows the maximum size and quantity of vertical reinforcement permitted by Sections 6.1.3.2.4, 6.1.3.2.5.1, and 6.1.3.4.4 for two-celled, 12-in. (305 mm) long clay masonry units. The maximum reinforcement listed in both tables may be doubled at lap splice locations.

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Table 6.1.3.2.5.2: Maximum Vertical Reinforcement Percentages Based on Specified Wythe Thickness and 8-in. (203 mm) Length When Laid in Stack Bond [1,2]

Unit Shape	Laps Excluded	Laps Included
Flanged ends	1.4 %	2.8%
Jamb ends	1.7%	3.4%
Open ends	2.1%	4.2%

1 Two-celled hollow concrete masonry and hollow clay masonry

2 Limited to hollow concrete units from 6-in. (152 mm) nominal thickness to 12-in. (305 mm) nominal thickness and to hollow clay units from 4-in. (102 mm) nominal thickness to 8-in. (203 mm) nominal thickness.

3 For clay masonry units with a 6-in. (152 mm) vertical reinforcement module, multiply the tabular value by 0.8 to determine the maximum vertical reinforcement percentage per 6-in. (152 mm) length.

6.1.3.3 The maximum size of welded wire reinforcement shall be D31 (MD 200) for welded deformed wire reinforcement. The minimum diameter of welded wire reinforcement used in masonry shall be 0.10 in. (2.5 mm).

6.1.4 *Placement of bar and deformed wire reinforcement*

6.1.4.1 The clear distance between parallel bars or deformed wires shall not be less than the nominal diameter of the reinforcement, nor less than 1 in. (25.4 mm).

6.1.4.2 In columns and pilasters, the clear distance between vertical bars and deformed wire shall not be less than one and one-half multiplied by the nominal reinforcement diameter, nor less than $1^1/_2$ in. (38.1 mm).

COMMENTARY

6.1.4 *Placement of bar and deformed wire reinforcement*

Placement limits for reinforcement are based on successful construction practice over many years. The limits are intended to facilitate the flow of grout between bars and between wires. A minimum spacing between bars in a layer prevents longitudinal splitting of the masonry in the plane of the bars. The prohibition on bundled bars and deformed wires stems from the lack of research on masonry with bundled reinforcement. It is important that bars and deformed wires be placed accurately. Reinforcing bar positioners are available to control bar position.

Requirements for the placement of mechanical splices are provided in Section 6.1.7.2.3.

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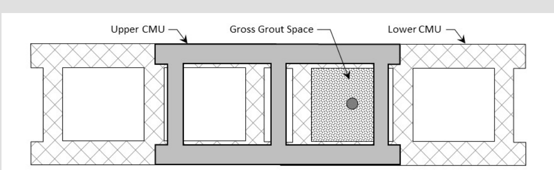
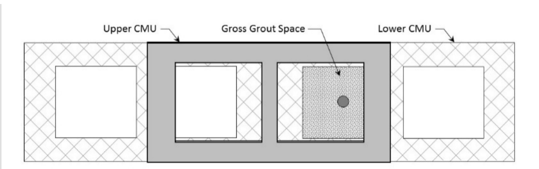
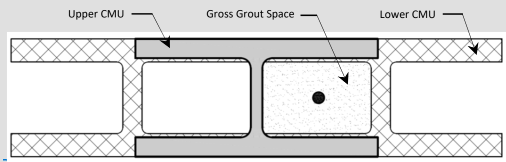


~~(a) Flanged units laid in one-half running bond~~



~~(b) Jamb units laid in one-half running bond~~



~~(c) Open-end units laid in one-half running bond~~

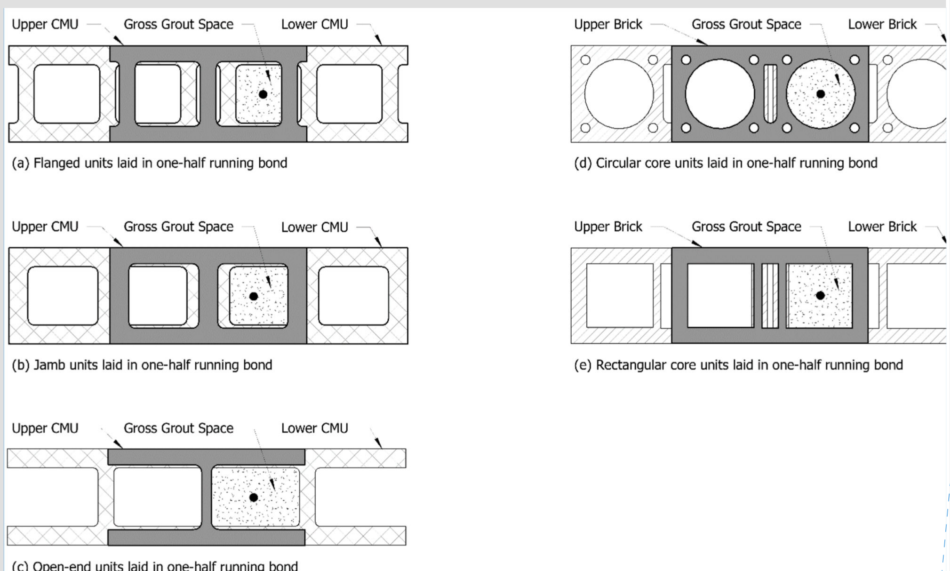


Figure CC-6.1-1 – Two-celled flanged units, jamb units, and open-end units laid in one-half running bond for concrete masonry units (a), (b), (c), and clay units (d), (e)

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Table CC-6.1.3.2.5.2.1: Maximum Vertical Reinforcement* per Alternative Provision – One-Half Running Bond Two-Celled Hollow Concrete Masonry and Hollow Clay Masonry

Nominal Unit Thickness	Maximum Vertical Reinforcement in 8 in. (203 mm) Wythe Length (per cell)		
	Flanged Units	Jamb Units	Open-End Units
6 in. (152 mm)	one #6 or two #4	one #6 or two #5	one #6 or two #5
8 in. (203 mm)	one #7 or two #5	one #8 or two #6	one #8 or two #6
10 in. (254 mm)	one #8 or two #6	one #9 or two #6	one #10 or two #7
12 in. (305 mm)	one #9 or two #6	one #10 or two #7	one #11 or two #8

*Reinforcement listed may be doubled at lap splice locations.

Table CC-6.1.3.2.5.2.2: Maximum Vertical Reinforcement* per Alternative Provision – Stack Bond Two-Celled Hollow Concrete Masonry and Hollow Clay Masonry

Nominal Unit Thickness	Maximum Vertical Reinforcement in 8 in. (203 mm) Wythe Length (per cell)		
	Flanged Units	Jamb Units	Open-End Units
6 in. (152 mm)	one #6 or two #5	one #6 or two #5	one #6 or two #6
8 in. (203 mm)	one #8 or two #6	one #8 or two #6	one #8 or two #7
10 in. (254 mm)	one #9 or two #6	one #9 or two #6	one #10 or two #8
12 in. (305 mm)	one #10 or two #7	one #11 or two #8	one #11 or two #8

*Reinforcement listed may be doubled at lap splice locations.

Table CC-6.1.3.2.5.2.3: Maximum Vertical Reinforcement* per Alternative Provision – Nominal 12 in. (305 mm) Long Hollow Clay Masonry

Nominal Unit Thickness	Maximum Vertical Reinforcement in 6 in. (152 mm) Wythe Length (per cell)	
	One-Half Running Bond	Stack Bonds
4 in. (102 mm)	one #4 or two #3	one #4 or two #3
5 in. (127 mm)	one #5 or two #3	one #5 or two #3
6 in. (152 mm)	one #5 or two #3	one #6 or two #4
8 in. (305 mm)	one #6 or two #4	one #7 or two #4

*Reinforcement listed may be doubled at lap splice locations.

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6.1.4.3 The clear distance limitations between bars and between deformed wires required in Sections 6.1.4.1 and 6.1.4.2 shall also apply to the clear distance between a contact lap splice and adjacent splices or reinforcement.

6.1.4.4 Bars and deformed wire reinforcement shall not be bundled.

6.1.4.5 Reinforcement embedded in grout shall have a thickness of grout between the reinforcement and masonry units not less than $^{1}/_{4}$ in. (6.4 mm) for fine grout or $^{1}/_{2}$ in. (12.7 mm) for coarse grout.

6.1.5 *Protection of reinforcement*

6.1.5.1 Reinforcement placed in grout shall have a masonry cover not less than the following:

(a) Masonry face exposed to earth or weather: 2 in. (50.8 mm) for bars larger than No. 5 (M #16); $1^{1}/_{2}$ in. (38.1 mm) for deformed wire, welded wire reinforcement, and No. 5 (M #16) bars or smaller.

(b) Masonry not exposed to earth or weather: $1^{1}/_{2}$ in. (38.1 mm).

6.1.5.2 Longitudinal wires of joint reinforcement and deformed wire placed in mortar joints shall be fully embedded in mortar or grout with a minimum cover of $^{5}/_{8}$ in. (15.9 mm) when exposed to earth or weather and $^{1}/_{2}$ in. (12.7 mm) when not exposed to earth or weather. Joint reinforcement and deformed wire placed in mortar joints 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 and deformed wire placed in mortar joints shall be mill galvanized, hot-dip galvanized, or stainless steel.

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6.1.5 *Protection of reinforcement*

6.1.5.1 Reinforcing bars and deformed wires embedded in grout are traditionally not coated for corrosion resistance. The masonry cover retards corrosion of the steel. Cover is measured from the exterior masonry surface to the outermost surface of the reinforcement to which the cover requirement applies. It is measured to the outer edge of stirrups or lateral ties, if transverse reinforcement encloses the longitudinal reinforcement. Masonry cover includes the thickness of masonry units, mortar, and grout. At bed joints, the protection for reinforcement is the total thickness of mortar and grout from the exterior of the mortar joint surface to outermost surface of the reinforcement or metal accessory.

The condition “masonry face exposed to earth or weather” refers to direct exposure to moisture changes (alternate wetting and drying) and not just temperature changes.

6.1.5.2 Because masonry cover protection for joint reinforcement is minimal, the protection of joint reinforcement in masonry is required in accordance with TMS 602. Examples of interior walls exposed to a mean relative humidity exceeding 75 percent are natatoria and food processing plants.

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6.1.6 *Development*

6.1.6.1 *Reinforcement in tension or compression* — The required tension or compression reinforcement shall be developed on each side of the critical section by development length, hook, mechanical device, or combination thereof. See Section 6.1.10 for critical sections.

6.1.6.2 *Reinforcement in mortar*

6.1.6.2.1 *Deformed wire reinforcement*

6.1.6.2.1.1 The development length of deformed wire embedded in mortar and subject to tension shall be determined by Equation 6-1, but shall not be less than 8 in. (203 mm).

$$\ell_d = 48\, d_b \qquad \text{(Equation 6-1)}$$

Development length of epoxy-coated deformed wire embedded in mortar and subject to tension shall be taken as 150 percent of the length determined by Equation 6-1.

6.1.6.2.1.2 Deformed wire embedded in mortar and used to resist applied vertical and lateral loads shall be embedded in mortar that complies with TMS 602 Article 2.1 A and utilizes either mortar cement or non-air-entrained portland cement and lime. This mortar limitation shall not apply when deformed wire is

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6.1.6 *Development*

6.1.6.1 *Reinforcement in tension or compression* — From a point of peak stress in reinforcement, some length of reinforcement or anchorage is necessary through which to develop the stress. This development length or anchorage is necessary on both sides of such peak stress points, on one side to transfer stress into and on the other to transfer stress out of the reinforcement. Often the reinforcement continues for a considerable distance on one side of a critical stress point so that calculations need involve only the other side; for example, the negative moment reinforcement continuing through a support to the middle of the next span.

6.1.6.2 *Reinforcement in mortar*

6.1.6.2.1 *Deformed wire reinforcement*

6.1.6.2.1.1 Equation ~~6-3~~ 6-1 was derived from the historic development length expression using an allowable bond stress u of 160 psi (1103 kPa), which was based on testing of deformed bars in grout (Gallagher (1935); Richart (1949)). ~~The 8 in. (203 mm) limit is based on research on joint reinforcement, which is the only research available for reinforcement placed in mortar. The research on joint reinforcement is discussed in the commentary to Section 6.1.7.1.1.2.. Research (Treece and Jirsa (1989)) has shown that epoxy-coated reinforcing bars require longer development length than uncoated reinforcing bars. The Committee deemed it appropriate to apply this same increase to deformed wires embedded in mortar. The 50 percent increase in development length does not apply to the 8 in. (203 mm) minimum.~~ Equation 6-1 was derived as follows:

$$\ell_d = F_s\, d_b / 4u = F_s\, d_b / 4(160) = 0.0015\, F_s\, d_b$$

$$(\ell_d = 0.22\, F_s\, d_b \text{ in SI units})$$

The term $0.0015 F_s d_b$ is equivalent to 45 d_b when F_s = 30,000 psi (206.84 MPa). The value was rounded up to 48 d_b to be consistent with other sections of the Code.

The 8 in. (203 mm) limit is based on research on joint reinforcement, which is the only research available for reinforcement placed in mortar. The research on joint reinforcement is discussed in the Code Commentary Section 6.1.6.2.2. Research (Treece and Jirsa (1989)) has shown that epoxy-coated reinforcing bars require longer development length than uncoated reinforcing bars. The Committee deemed it appropriate to apply this same increase to deformed wires embedded in mortar. The 50 percent increase in development length does not apply to the 8 in. (203 mm) minimum.

6.1.6.2.1.2 The limitations in this section are based on the limitations of the research performed on joint reinforcement, as discussed in the commentary to Section 6.1.~~7.1.1~~6.2.2.

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only used to resist masonry volume changes, the masonry is designed with movement joints, and the deformed wire is not used to resist applied vertical or lateral loads.

6.1.6.2.2 *Joint reinforcement*

6.1.6.2.2.1 The minimum development length for joint reinforcement shall be 8 in. (203 mm).

6.1.6.2.2.2 Joint reinforcement used to resist applied vertical and lateral loads shall be embedded in mortar that complies with TMS 602 Article 2.1 A and utilizes either mortar cement or non-air-entrained portland cement and lime. This mortar limitation shall not apply when joint reinforcement is only used as a tie between wythes or to resist masonry volume changes when the masonry is designed with movement joints, and the joint reinforcement is not used to resist applied vertical or lateral loads.

6.1.6.3 *Bar, deformed wire reinforcement, and welded deformed wire reinforcement in grout*

6.1.6.3.1 *Clay masonry and concrete masonry* — The required development length of reinforcing bars, deformed wires, and welded deformed wire reinforcement embedded in grout shall be determined by Equation 6-2, but shall not be less than 12 in. (305 mm).

$$\ell_d = \frac{0.13 d_b^2 f_y \gamma}{K\sqrt{f'_m}} \qquad \text{(Equation 6-2)}$$

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

γ = 1.0 for No. 3 (M#10) through No. 5 (M#16) bars

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6.1.6.2.2 *Joint reinforcement* – The longitudinal wires of joint reinforcement are fully developed to their tensile strength when embedded 8 in. (203 mm) in portland cement/lime mortar that is not air entrained or in mortar cement mortar, according to research by Saxer (1956). Saxer's research on development of joint reinforcement wires did not consider masonry cement mortar and air-entrained portland cement/lime mortar, so those mortar materials are not permitted when the joint reinforcement is used to resist applied loads. The wire's tensile strength need not be fully developed when the joint reinforcement only needs to resist masonry volume changes in masonry that is designed with movement joints or when it is only used as a connector between wythes and, therefore, any cementitious material permitted by TMS 602 Article 2.1 A may be used in mortar that embeds joint reinforcement used for those purposes.

Development and lap splicing of joint reinforcement subjected to cyclic loading and to cracking of the mortar joints have neither been studied by either Saxer (1956) or Baenziger and Porter (2011). Further research is required to determine whether the 8-inch (203 mm) specified development length and lap splice length are adequate for those conditions.

6.1.6.3 *Bar, deformed wire reinforcement, and welded deformed wire reinforcement in grout*

6.1.6.3.1 *Clay masonry and concrete masonry* — The clear spacing between adjacent reinforcement does not apply to the reinforcement being spliced. Refer to Commentary 6.1.7.1.2.1 for further information.

The factor K in Equation 6-2 accounts for the beneficial effect of increased reinforcement cover and spacing in delaying the onset of splitting of the masonry leading to pull out of the reinforcement. The $9d_b$ limit on K is based on studies by Schultz (2004, 2005) as well as testing and subsequent analysis by the National Concrete Masonry Association (2009). NCMA (2009) recommended that K be limited to $8.8d_b$, which is rounded to the current $9d_b$ limit. When cover and spacing

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and deformed wires;

γ = 1.3 for No. 6 (M#19) through No. 7 (M#22) bars;

and

γ = 1.5 for No. 8 (M#25) and larger bars.

Development length of epoxy-coated bars and deformed wires embedded in grout shall be taken as 150 percent of the length determined by Equation 6-2.

6.1.6.3.2 *AAC masonry* — The required development length of reinforcement shall be determined by Section 6.1.6.3.1, and replacing f'_m with f'_g and K with K_{AAC} in Equation 6-2. K_{AAC} shall not exceed the smallest of the following: the minimum grout cover, the clear perpendicular spacing between adjacent reinforcement splices, and $9d_b$.

6.1.6.3.3 *Standard hooks* — ~~Standard hooks~~ The required development length ℓ_{dh} of bars and deformed wires terminating in a standard hook in grout subject to tension shall be ~~considered to develop an equivalent embedment length, ℓ_e, as~~ determined by Equation 6-3. Hooks shall not be used to develop bars or deformed wires in compression.

$$\cancel{\ell_e = 13\,d_b}\ \ell_{dh} = \ell_d - \gamma_h\, d_b \qquad \text{(Equation 6-3)}$$

γ_h = 9.0 for No. 3 (M#10) through No. 8 (M#25) bars and deformed wires; and

γ_h = 8.0 for No. 9 (M#29) through No. 11 (M#36) bars

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exceeds this amount, splitting is not expected to occur and there is no benefit from additional cover or spacing.

Refer to Figure CC-6.1-2 for the clear perpendicular spacing between adjacent reinforcement splices.

Due to lack of experimental data on the development of welded deformed wires in grout, the development length is determined without consideration of the beneficial effects of welded cross wires

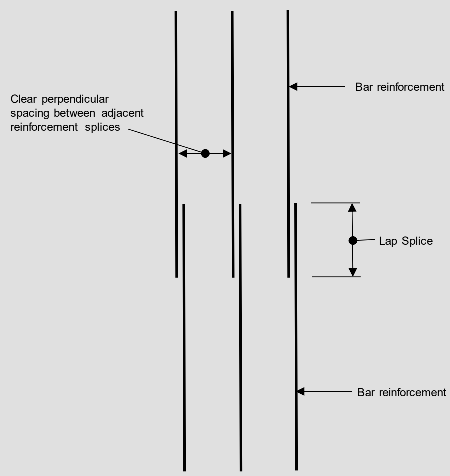

Figure CC-6.1-2 — Clear perpendicular spacing between adjacent reinforcement splices

The 50 percent increase in development length for epoxy-coated bars does not apply to the 12 in. (305 mm) minimum.

6.1.6.3.2 *AAC masonry* — Development and lap splice detailing provisions for conventional masonry are calibrated to the masonry assembly strength, f'_m, which includes the contribution of each constituent material (unit, grout, and mortar). Due to the low compressive strength of AAC, however, the AAC masonry component is ignored and the calibration is based on f'_g.

6.1.6.3.3 *Standard hooks* — Historically, standard hooks were considered to be able to develop a stress in the bar or wire of 7,500 psi (51.72 MPa). The remainder of the stress in the bar due to design loads was required to be developed in bond along the straight length of bar starting at the tangent point of the hook. When the bond stress model for development of bars was replaced by Equation 6-2, the 7,500 psi (51.72 MPa) was converted into an equivalent embedment length of $13d_b$. The minimum distance from the point where the bar needed to be developed to the tangent point of the hook, was determined by subtracting $13d_b$ from Equation 6-1 or Equation 6-2. Equation 6-3 now defines a hooked development length, ℓ_{dh}, in a manner consistent with ACI 318. The γ_h factor in

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Equation 6-3 was determined by subtracting from $13d_b$ the inside radius of hook determined from TMS 602 Table 6 and one bar or wire diameter, resulting in a value for ℓ_{dh} measured to the outside of the bar at the hook. This is illustrated in Figure CC-6.1-3.

When compared to the hooked development length equation in ACI 318, Equation 6-3 suggests that hooks are less effective in masonry than in concrete. This is likely an artifact of the historical basis of Equation 6-3, and is not indicative of a fundamental difference between concrete and masonry. A more refined and potentially less conservative equation for ℓ_{dh} is anticipated to be developed for a future edition of this Code.

In compression, hooks are ineffective and cannot be used as anchorage.

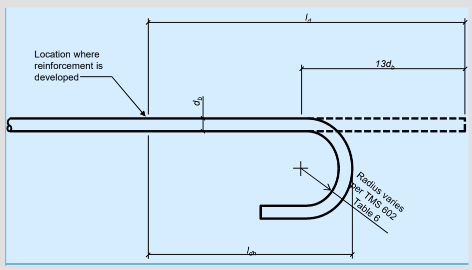

Figure CC-6.1-3 — Hooked development length

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6.1.7 *Splices*

6.1.7 *Splices* — Continuity of reinforcement through proper splicing is necessary to provide force transfer. Effective splices can be provided through various forms: lap splices, welded splices or mechanical splices.

6.1.7.1 *Lap splices*.

6.1.7.1 *Lap splices*

6.1.7.1.1 *Reinforcement embedded in mortar*

6.1.7.1.1 *Reinforcement embedded in mortar*

6.1.7.1.1.1 *Deformed wire reinforcement* — The minimum length of lap for deformed wires embedded in mortar and subject to tension shall be determined by Equation 6-1, but shall not be less than 8 in. (203 mm).

6.1.7.1.1.1 *Deformed wire reinforcement*

6.1.7.1.1.2 *Joint reinforcement* – Lap splices of joint reinforcement shall comply with the development requirements of Section 6.1.6.2.2.

6.1.7.1.1.2 *Joint reinforcement* – Refer to commentary for Section 6.1.6.2.2.

6.1.7.1.2 *Bar and deformed wire reinforcement embedded in grout*

6.1.7.1.2 *Bar and deformed wire reinforcement embedded in grout*

6.1.7.1.2.1 The minimum length of lap for bars, deformed wires, and welded deformed wire reinforcement embedded in grout and subject to tension or compression shall be determined by Section 6.1.6.3.1 for clay masonry and concrete masonry and by Section 6.1.6.3.2 for AAC masonry, but not less than 12 in. (305 mm).

6.1.7.1.2.1 The required length of the lap splice is based on developing a minimum reinforcing bar stress of 1.25 f_y. This is conservative for deformed wires whose minimum specified tensile strength (85 ksi) is less than 1.15 times the minimum specified yield strength (75 ksi). This requirement provides adequate strength while maintaining consistent requirements between lap, mechanical, and welded splices. Historically, the length of lap has been based on the bond stress that is capable of being developed between the reinforcement and the surrounding grout. Testing has shown that bond failure (or pull-out of the reinforcement) is only one possible mode of failure for lap splices. Other failure modes include rupture of the reinforcement and longitudinal splitting of masonry along the length of the lap. Experimental results of several independent research programs were combined and analyzed to provide insight into predicting the necessary lap lengths for reinforcing bar splices in masonry construction (Hogan et al (1997)). Equation 6-2 was fitted to the data and has a coefficient of determination, r^2, value of 0.93.

Due to lack of experimental data on the splicing of welded deformed wires in grout, the splice length is determined without consideration of the beneficial effects of welded cross wires.

6.1.7.1.2.2 For clay masonry and concrete masonry, where reinforcement with a diameter greater than 0.35-in. (9-mm) is placed transversely within the lap, with at least one bar or deformed wire 8 in. (203 mm) or less from each end of the lap, the minimum length of lap for bars, deformed wires, and welded deformed wire reinforcement in tension or compression determined by Equation 6-2 shall be permitted to be reduced by multiplying by the confinement factor, ξ, determined in accordance with Equation 6-4. The clear space between the transverse reinforcement and the lapped reinforcement shall not exceed 1.5 in. (38 mm) and the transverse reinforcement shall be fully developed in grouted masonry. The reduced lap splice length shall not be less than $36d_b$.

6.1.7.1.2.2 An extensive testing program conducted by the National Concrete Masonry Association (NCMA (2009)) and additional testing done by Washington State University (Mjelde et al (2009)) show that reinforcement provided transverse to lapped reinforcement controls longitudinal tensile splitting of the masonry assembly. These transverse bars increase the lap performance significantly, as long as there is at least one No. 3 (M#10) transverse reinforcing bar placed within 8 in. (203 mm) of each end of the splice. These bars must be fully developed using straight reinforcement of adequate length or hooks as depicted in Figure CC-~~6.1-3~~6.1-4, and have a clear spacing between the transverse reinforcement and the lapped reinforcement not exceeding 1.5 in. (38 mm). Testing also indicated that the lap length must be

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$$\xi = 1.0 - \frac{2.3A_{sc}}{d_b^{2.5}} \qquad \text{(Equation 6-4)}$$

$$\text{Where : } \frac{2.3A_{sc}}{d_b^{2.5}} \leq 1.0$$

A_{sc} is the area of the transverse reinforcement at each end of the lap splice and shall not be taken greater than 0.35 in.^{2} (226 mm^2).

6.1.7.1.2.3 Reinforcement spliced by noncontact lap splices shall not be spaced transversely farther apart than one-fifth the length of lap nor more than 8 in. (203 mm). Noncontact splices are not permitted in AAC masonry.

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at least 36d_b or the effect of the transverse reinforcement is minimal. As a result, this limit was applied to the lap length. The testing also showed that even when more transverse reinforcement area is provided, it becomes significantly less effective in quantities above 0.35 in.^{2} (226 mm^2). Thus, the transverse reinforcement area at each end of the lap, A_{sc}, is limited to 0.35 in.^{2} (226 mm^2), even if more is provided.

6.1.7.1.2.3 If individual bars or deformed wires in noncontact lap splices are too widely spaced, an unreinforced section is created, which forces a potential crack to follow a diagonal line. Lap splices in clay masonry and concrete masonry may occur with the reinforcement in adjacent grouted cells if the requirements of this section are met. Lap lengths may need to exceed the minimum calculated in accordance with Section 6.1.7.1.2.1 to permit offsets into adjacent cells (for vertical bars or deformed wires) or adjacent courses (for horizontal bars or deformed wires, as would be required for stepped bond beams).

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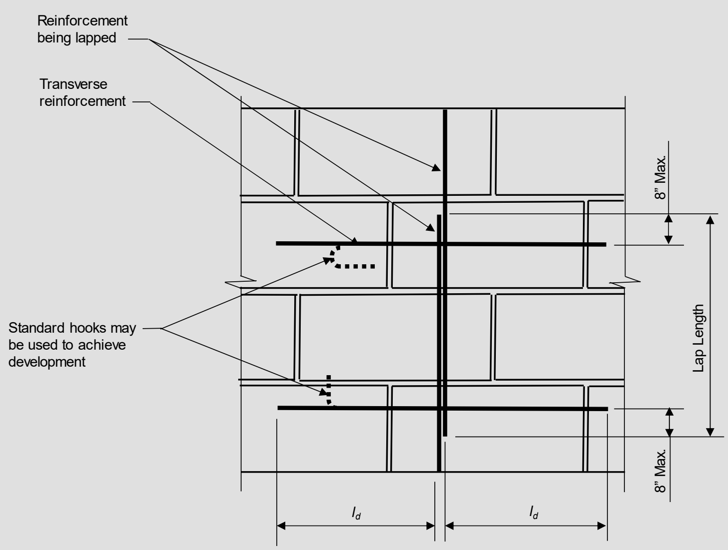

Figure CC-~~6.1-3~~6.1-4 — Confining transverse reinforcement at lap lplice

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6.1.7.2 *Mechanical splices*

6.1.7.2.1 *Bar reinforcement* — 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.

6.1.7.2.2 *Deformed wire reinforcement* — Mechanical splices shall have the deformed wires connected to develop the specified tensile strength of the wire. Mechanical splices shall not be used for deformed wire placed in mortar.

6.1.7.2.3 *Size and placement* — Mechanical splices shall meet the following additional requirements:

(a) The greatest cross-sectional dimension of the mechanical splice shall not exceed one-third of the least dimension of the gross grout space in which it is placed.

(b) The cross-sectional area of the mechanical splice shall be treated as lapped reinforcement for the purpose of determining compliance with Section 6.1.3.2.5.

(c) The clear distance limitations between bars and between deformed wires required in Sections 6.1.4.1 and 6.1.4.2 shall also apply to the clear distance between a mechanical splice and adjacent splices or reinforcement. For the purpose of this provision, consider the nominal diameter of the splice to be the greatest cross-sectional dimension of the mechanical splice.

(d) The thickness of grout between the mechanical splice and the masonry units shall comply with Section 6.1.4.5.

(e) The mechanical splice shall have a masonry cover of 2 in. (50.8 mm) from any masonry face exposed to earth or weather and 1 ½ in. (38.1 mm) from all other masonry faces.

6.1.7.3 *Welded splices*

6.1.7.3.1 *Bar reinforcement* — Welded splices of reinforcing bars shall have the bars butted and welded to develop at least 125 percent of the specified yield strength, f_y, of the bar in tension or compression, as required. 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

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6.1.7.2 *Mechanical splices*

6.1.7.2.1 *Bar reinforcement* — ~~Full m~~Mechanical splices are also required to develop 125 percent of the specified yield strength in tension or compression as required, for the same reasons discussed for full welded splices.

6.1.7.2.2 *Deformed wire reinforcement* — Mechanical splices of deformed wire are required to develop the specified tensile strength of the deformed wire instead of 125 percent of the yield strength as is required for reinforcing bars because the minimum specified tensile strength (85 ksi) of ASTM A1064 deformed wire is less than 125 percent of the minimum specified yield strength (75 ksi). Mechanical couplers that have been developed and tested for reinforcing bars may not be suitable for deformed wires due to differences in yield strength and deformations. Mechanical splices of deformed wires in mortar is not permitted because the coupler does not fit in the mortar joint.

6.1.7.2.3 *Size and placement* — This section adapts the size limitations and placement requirements of Sections 6.1.3, 6.1.4 and 6.1.5 to mechanical splices, to maintain appropriate clearances for grouting and protection of the mechanical splice. If multiple bars are mechanically spliced in the same grout space, the splices may be staggered to achieve compliance with this section.

The references to cross-sectional area and greatest cross-sectional dimension of the mechanical splice are intended to refer to the main body of the splice, not to localized protrusions from the body such as bolt heads or 2022 TMS 402/602 Ballot Item 21-RC-007 Page 4 of 4 ports. Clear distance and cover requirements should be met at all portions of the coupler, including at localized protrusions.

6.1.7.3 *Welded splices*

6.1.7.3.1 *Bar reinforcement* — A full welded splice is primarily intended for large bars (No. 6 [M#19] and larger) in main members. The tensile strength requirement of 125 percent of specified yield strength is intended to ensure sound welding, adequate also for compression. It is desirable that splices be capable of developing the ultimate tensile strength of the bars spliced, but practical limitations make this ideal condition difficult to attain. The maximum reinforcement stress used in design

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shall be analyzed chemically and its carbon equivalent determined as required by AWS D1.4/D1.4M.

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under this Code is based upon yield strength. To ensure sufficient strength in splices so that brittle failure can be avoided, the 25 percent increase above the specified yield strength was selected as both an adequate minimum for safety and a practicable maximum for economy.

When welding of reinforcing bars is required, the weldability of the steel and compatible welding procedures need to be considered. The provisions in AWS D1.4/D1.4M Welding Code cover aspects of welding reinforcing bars, including criteria to qualify welding procedures. Weldability of the steel is based on its chemical composition or carbon equivalent (CE). The Welding Code establishes preheat and interpass temperatures for a range of carbon equivalents and reinforcing bar sizes. Carbon equivalent is calculated from the chemical composition of the reinforcing bars. The Welding Code has two expressions for calculating carbon equivalent. A relatively short expression, considering only the elements carbon and manganese, is to be used for bars other than ASTM A706 material. A more comprehensive expression is given for ASTM A706 bars. The CE formula in the Welding Code for ASTM A706 bars is identical to the CE formula in ASTM A706.

The chemical analysis, for bars other than ASTM A706, required to calculate the carbon equivalent is not routinely provided by the producer of the reinforcing bars. For welding reinforcing bars other than ASTM A706 bars, the design drawings or project specifications should specifically require results of the chemical analysis to be furnished.

ASTM A706 covers low-alloy steel reinforcing bars intended for applications requiring controlled tensile properties or welding. Weldability is accomplished in ASTM A706 by limits or controls on chemical composition and on carbon equivalent (Gustafson and Felder (1991)). The producer is required by ASTM A706 to report the chemical composition and carbon equivalent.

The AWS D1.4/D1.4M Welding Code requires the contractor to prepare written welding procedure specifications conforming to the requirements of the Welding Code. Appendix A of the Welding Code contains a suggested form that shows the information required for such a specification for each joint welding procedure.

Welding to existing reinforcing bars is often necessary even though no mill test report of the existing reinforcement is available. This condition is particularly common in alterations or building expansions. AWS D1.4/D1.4M states for such bars that a chemical analysis may be performed on representative bars. If the chemical composition is not known or obtained, the Welding Code requires a minimum preheat. Welding of the particular bars should be performed in accordance with AWS D1.4/D1.4M, including their preheat. The designer should also determine if additional precautions are in order, based on other considerations such as stress level in the bars, consequences of failure, and heat damage to existing masonry due to welding operations.

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6.1.7.3.2 *Deformed wire reinforcement and welded deformed wire reinforcement* — Welded splices of deformed wire and welded deformed wire reinforcement shall develop the specified tensile strength of the wire.

6.1.7.4 *End-bearing splices*

6.1.7.4.1 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.

6.1.7.4.2 Bar ends shall terminate in flat surfaces within $1^1/_2$ degree of a right angle to the axis of the bars and shall be fitted within 3 degrees of full bearing after assembly.

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

6.1.8 *Shear reinforcement*

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.

~~**6.1.8.1** *Horizontal shear reinforcement* — Horizontal reinforcement shall meet the requirements of Sections 6.1.8.1.1 through 6.1.8.1.3~~

~~**6.1.8.1.1** Except at wall intersections, the ends of horizontal reinforcing bar or deformed wire shall be bent around the edge vertical reinforcing bar or deformed wire with a 180-degree standard hook.~~

~~**6.1.8.1.2** At wall intersections, horizontal reinforcing bars or deformed wire shall be bent around the edge vertical reinforcing bar or deformed wire with a 90-degree standard hook and shall extend horizontally into~~

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6.1.7.1.3.2 *Deformed wire reinforcement and welded deformed wire reinforcement* — Welding of wire is not covered by AWS D1.4/D1.4M. If welding of wires is required on a project, the contract documents should specify requirements or performance criteria for the welding. If cold drawn wires are to be welded, the welding procedures should address the potential loss of yield strength and ductility achieved by the cold working process (manufacturing) when such wires are heated by welding. Machine and resistance welding, as used in the manufacture of welded plain and deformed wire reinforcement, are covered by ASTM A1064/A1064M and by ASTM A951 for joint reinforcement.

Welded splices of deformed wire are required to develop the specified tensile strength of the deformed wire instead of 125 percent of the yield strength as is required for reinforcing bars because the minimum specified tensile strength (85 ksi) of ASTM A1064 deformed wire is less than 125 percent of the minimum specified yield strength (75 ksi).

6.1.7.4 *End-bearing splices* — Experience with end-bearing splices has been almost exclusively with vertical bars in columns. If bars are significantly inclined from the vertical, special attention is required to ensure that adequate end-bearing contact can be achieved and maintained. The lateral tie requirements prevent end-bearing splices from sliding.

6.1.8 *Shear reinforcement*

Design and detailing of shear reinforcement locations and anchorage in masonry requires consideration of the masonry module and reinforcement cover and clearance requirements.

~~**6.1.8.1** *Horizontal shear reinforcement*~~ Given the definition of "shear reinforcement" in Section 2.2, the requirements of Section 6.1.8~~.1~~ only apply to ~~horizontal~~ shear reinforcement required by analysis. The requirements do not apply to other ~~horizontal~~ reinforcement, such as prescriptive reinforcement or crack-control reinforcement, although there may be other requirements for these bars.

~~**6.1.8.1.1** In a wall without an intersecting wall at its end, the edge vertical bar or deformed wire is the bar or deformed wire closest to the end of the wall.~~

~~**6.1.8.1.2** When the wall has an intersecting wall at its end, the edge vertical bar or deformed wire is the bar or deformed wire at the intersection of walls. Hooking the horizontal reinforcement around a vertical bar or~~

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~~the intersecting wall a minimum distance at least equal to the development length.~~

6.1.8.1 Deformed wire embedded in mortar and used as shear reinforcement shall be anchored by either:

(a) A 90-degree bend in longitudinal wires bent around the edge cell and with at least 3-in. (76-mm) bend extensions in mortar or grout, or

(b) A 90-degree bend in longitudinal wires bent around the edge cell and with at least 4-in. (102-mm) overlap of the wires in mortar or grout.

6.1.8.~~1.3~~2 Joint reinforcement used as shear reinforcement shall be anchored around the edge reinforcing bar or deformed wire in the edge cell~~, either by placement of the vertical reinforcement between adjacent cross-wires or with a 90-degree bend in longitudinal wires bent around the edge cell and with at least 3-in. (76-mm) bend extensions in mortar or grout~~ in accordance with either Section 6.1.8.~~1.3~~2.1 or 6.1.8.~~1.3~~2.2.

6.1.8.~~1.3~~2.1 Where the joint reinforcement consists of two longitudinal wires, both of the wires shall be anchored~~either~~ by one of the following:

(a) Placement of the vertical reinforcement between adjacent cross-wires, or

(b) A 90-degree bend in longitudinal wires bent around the edge cell and with at least 3-in. (76-mm) bend extensions in mortar or grout, or

(c) A 90-degree bend in longitudinal wires bent around the edge cell and with at least 4-in. (102-mm) overlap of the wires in mortar or grout.

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~~deformed wire located within the wall running parallel to the horizontal reinforcement would cause the reinforcement to protrude from the wall.~~

6.1.8.1 The options for the anchorage of deformed wire in mortar are based on the provisions for the anchorage of joint reinforcement - 6.1.8.1 (a) is equivalent to 6.1.8.2.1 (b) for joint reinforcement, and 6.1.8.1 (b) is equivalent to 6.1.8.2.1 (c) for joint reinforcement. The joint reinforcement options in Section 6.1.8.2.1 are depicted in Figure CC-6.1-5; deformed wire would appear the same except that no cross wire would be present.

6.1.8.1 (b) is intended for use in applications where enhanced ductility is desirable. As discussed in the Code Commentary Section 6.1.8.2.1, testing of the detail in four-wire joint reinforcing suggests it provides ductility suitable for use in special reinforced masonry shear walls.

6.1.8.~~1.3~~2 Wire reinforcement should be anchored around or beyond the edge reinforcing bar or deformed wire. Joint reinforcement longitudinal wires and wire bends are placed over masonry unit face shells in mortar and wire extensions can be placed in edge cell mortar or can extend into edge cell grout. Both joint reinforcement longitudinal wires and cross wires can be used to confine vertical reinforcing bars and deformed wires and grouted cells because wires are developed within a short length.

6.1.8.~~1.3~~2.1 The options described for anchoring joint reinforcement are illustrated in Figure CC-~~6.1-4~~6.1-5. Option (a) was used in the testing performed by Baenziger and Porter (2018) and demonstrated performance adequate for use in special reinforced masonry shear walls. While option (c) was not used in the testing, the good performance of overlapped wires in the four wire specimens demonstrated the adequacy of this detail. Option (b) has not been tested for use in special reinforced masonry shear walls.

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6.1.8.~~1.3~~2.2 Where the joint reinforcement consists of four longitudinal wires, all four of the wires shall be anchored by either:

(a) A 90-degree bend in the inner longitudinal wires bent around the edge cell and with at least 3-in. (76-mm) bend extensions in mortar or grout, and a 3/16 in. (5 mm) U-stirrup lapped at least 8-in. (205-mm) with the outer wires, or

(b) A 90-degree bend in both the inner and outer longitudinal wires bent around the edge cell and with at least 4-in. (102-mm) overlap of the wires in mortar or grout.

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6.1.8.~~1.3~~2.2 The options described for anchoring joint reinforcement are illustrated in Figure CC-~~6.1-4~~6.1-6. Both options were used in the testing performed by Baenziger and Porter (2018) and demonstrated performance adequate for use in special reinforced masonry shear walls.

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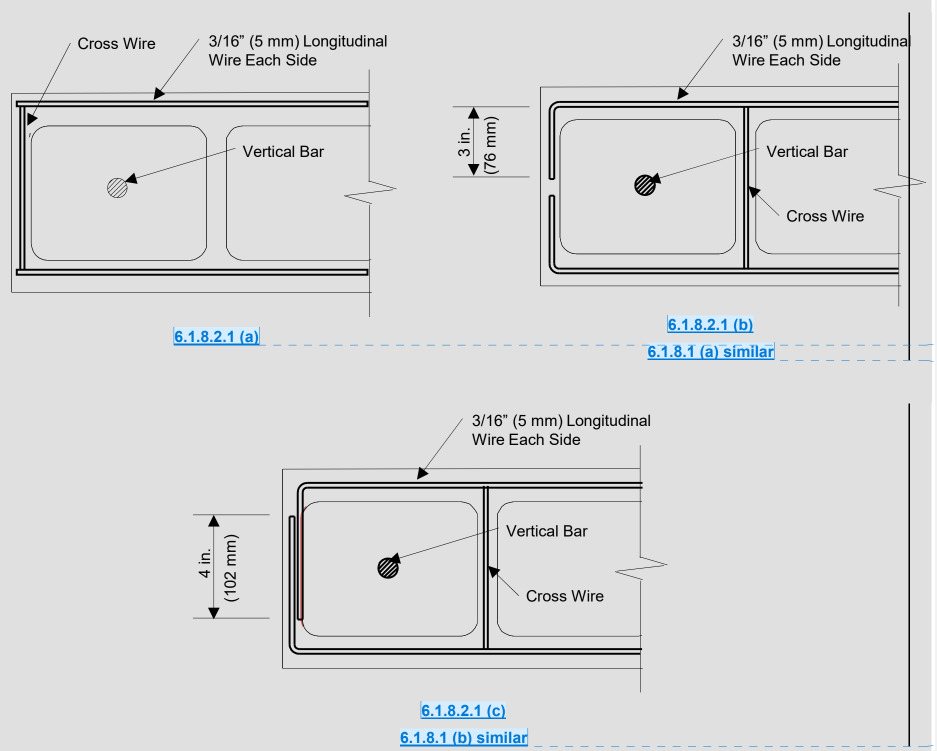

Figure CC-~~6.1-4~~6.1-5 — Anchorage of joint reinforcement with two longitudinal wires

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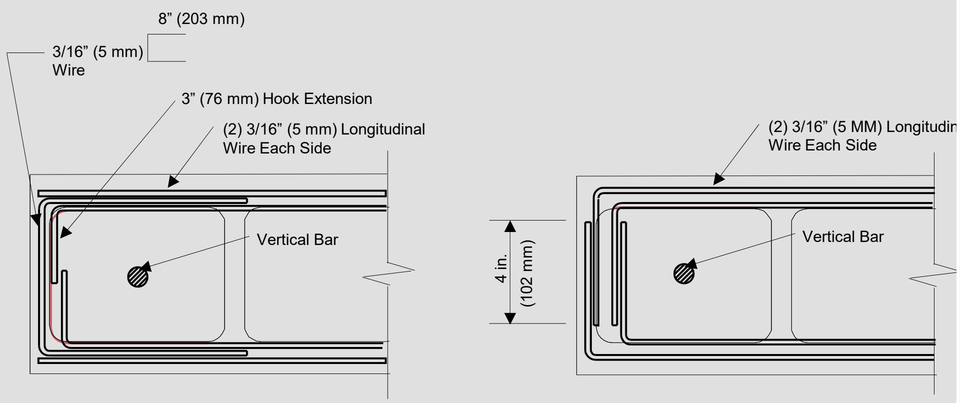

Figure CC-~~6.1-5~~6.1-6 — Anchorage of joint reinforcement with four longitudinal wires

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6.1.8.2 *Stirrups*

6.1.8.2.1 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 0.5 ℓ_d. The effective embedment of a stirrup leg shall be taken as the distance between the middepth of the member, $d/2$, and the start of the hook (point of tangency).

(b) For No. 5 bar (M #16) and D31 (MD200) wire and smaller, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of 0.33 ℓ_d. The 0.33 ℓ_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).

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6.1.8.2 *Stirrups*

Stirrups must be carried as close to the compression face of the member as possible because near ultimate load, flexural tension cracks penetrate deeply.

6.1.8.2.1 The requirements for anchorage of U-stirrups for deformed reinforcing bars and deformed wire are illustrated in Figure CC-~~6.1-6~~6.1-7.

(a) When a standard hook is used, 0.5 ℓ_d must be provided between $d/2$ and the point of tangency of the hook.

This provision may require a reduction in size and spacing of web reinforcement, or an increase in the effective depth of the beam, for web reinforcement to be fully effective.

(b) U-stirrups that enclose longitudinal reinforcement have sufficient pullout resistance in the tension zone of the masonry.

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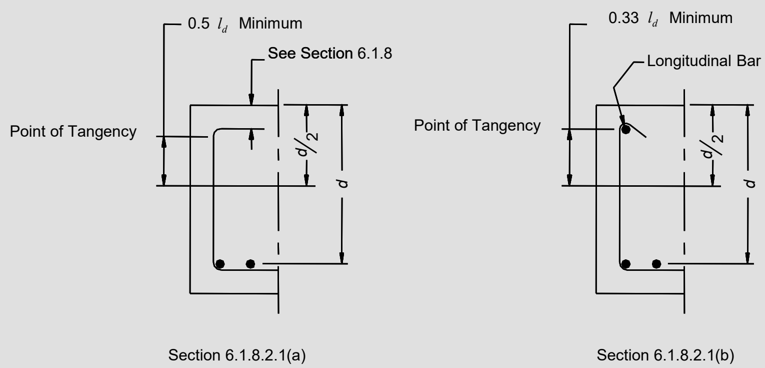

Figure CC-~~6.1-6~~6.1-7 — Anchorage of U-stirrups (deformed reinforcing bars and deformed wire)

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

6.1.8.2.2 Pairs of U stirrups or lateral ties placed to form a closed unit shall be considered properly spliced when length of laps are 1.7 ℓ_d . In grout at least 18 in. (457 mm) deep, such splices with $A_v f_y$ not more than 9,000 lb (40030 N) per leg shall be permitted to be considered adequate if legs extend the full available depth of grout.

6.1.8.3.3 *Longitudinal bent bars and deformed wire* — Longitudinal bars and deformed wire 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 middepth of the member, d/2.

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6.1.9 *Standard hooks and bends for reinforcing bars and deformed wire*

6.1.9.1 Standard hooks shall be as described in TMS 602 Table 6.

6.1.9.2 The diameter of bend measured on the inside of deformed wire of at least size D11 and reinforcing bars shall not be less than values specified in TMS 602 Table 6.

6.1.9.3 The diameter of bend measured on the inside of deformed wire larger than size D6 and smaller than size D11 shall not be less than $4d_b$.

6.1.9.4 The diameter of bend measured on the inside of deformed wire up to and including size D6 shall not be less than $2d_b$.

6.1.10 *Embedment of flexural reinforcement*

6.1.10.1 *General*

6.1.10.1.1 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.

6.1.10.1.2 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.

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6.1.9 *Standard hooks and bends for reinforcing bars and deformed wire*

The concrete building code (ACI 318-19) treats deformed wire and bars identically in the range of 3/8 in. (9.5 mm) diameter to 5/8 in. (15.9 mm) diameter because testing has shown consistent behavior with the two materials. Consequently, this Code provides the same requirements for hook and bends of bars and deformed wires in that size range. This consistency is supported by a comparison of the deformation requirements of ASTM A1064 and ASTM A615, which show substantial similarity in the required geometry.

6.1.10 *Embedment of flexural reinforcement* — Figure CC-~~6.1-7~~6.1-8 illustrates the embedment requirements of flexural reinforcement in a typical continuous beam. Figure CC-~~6.1-8~~6.1-9 illustrates the embedment requirements in a typical multi-span wall that is not part of the lateral-force-resisting system.

6.1.10.1 *General*

6.1.10.1.2 Critical sections for a typical continuous beam are indicated in Figure CC-~~6.1-7~~6.1-8. Critical sections for a multi-span wall are indicated in Figure CC-~~6.1-8~~6.1-9.

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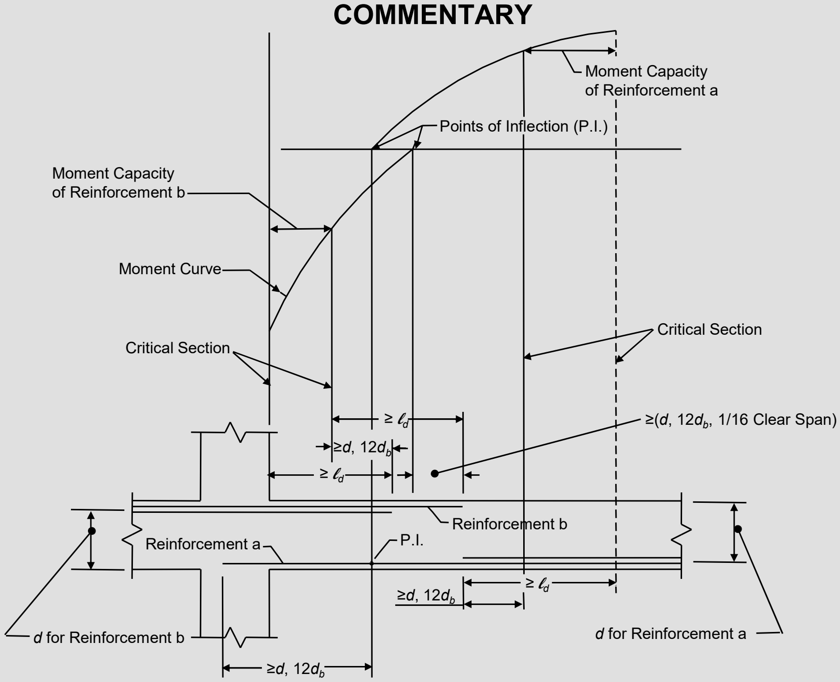

Figure CC-~~6.1-7~~6.1-8 — Development of flexural reinforcement in a continuous beam

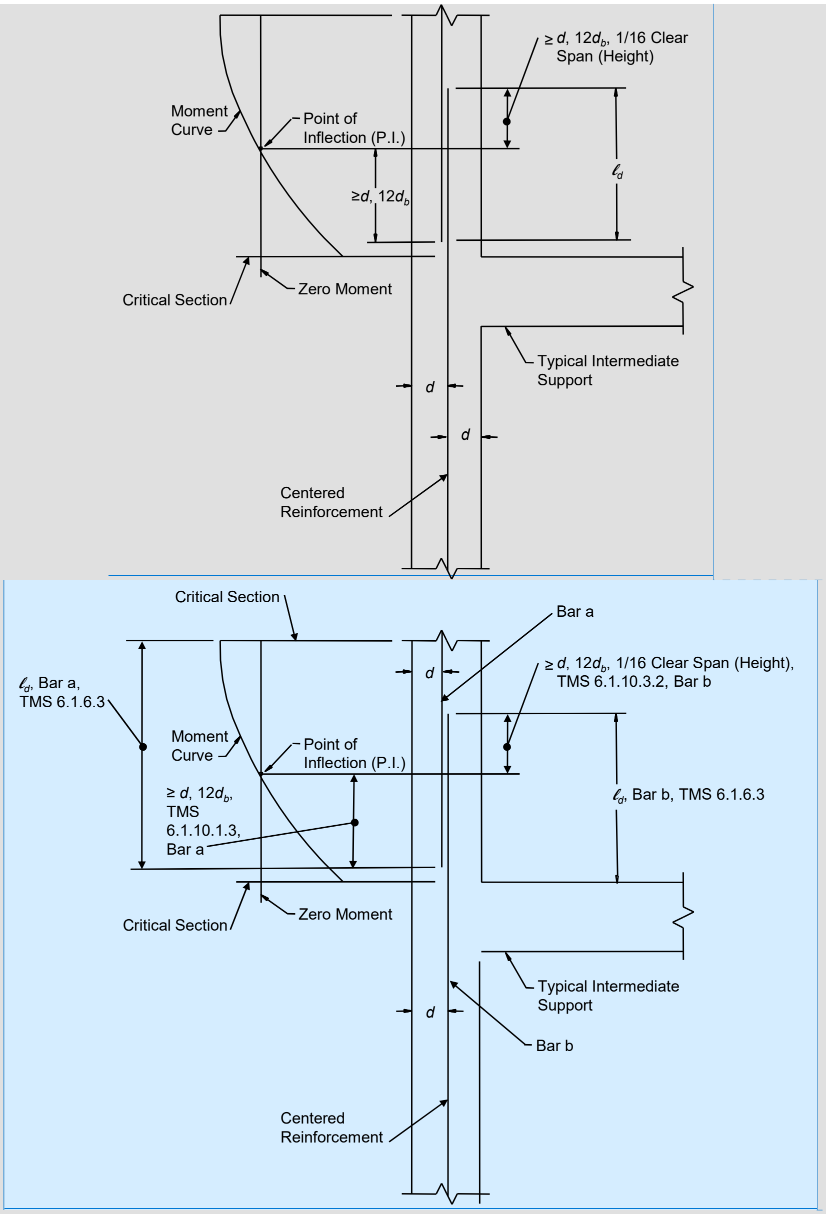

Figure CC-~~6.1-8~~6.1-9 — Development of flexural reinforcement in a continuous wall with centered reinforcement

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6.1.10.1.3 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.

6.1.10.1.4 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 6.1.6.1 or 6.1.6.2.

6.1.10.1.5 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:

(i) the allowable shear at the section considered, when using Chapter 8; or,

(ii) the design strength in shear at the section considered, when using Chapter 9 or Chapter 11.

(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 $60\, b_w s/f_y$. 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:

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6.1.10.1.3 The moment diagrams customarily used in design are approximate. Some shifting of the location of maximum moments may occur due to changes in loading, settlement of supports, lateral loads, or other causes. A diagonal tension crack in a flexural member without stirrups may shift the location of the calculated tensile stress approximately a distance d toward a point of zero moment. When stirrups are provided, this effect is less severe, although still present.

To provide for shifts in the location of maximum moments, this Code requires the extension of reinforcement a distance d or $12d_b$ beyond the point at which it is theoretically no longer required to resist flexure, except as noted. In lieu of providing the development lengths and bar extensions shown in Figure CC-6.1-8, the reinforcing may be made continuous with a splice as defined in Section 6.1.7.

Cutoff points of bars or deformed wires to meet this requirement are illustrated in Figure CC-~~6.1-7~~6.1-8.

When bars or deformed wires of different sizes are used, the extension should be in accordance with the diameter of reinforcement being terminated. A bar or deformed wire bent to the far face of a beam and continued there may logically be considered effective in satisfying this section, to the point where the bar or deformed wire crosses the middepth of the member.

6.1.10.1.4 Peak stresses exist in the remaining bars or deformed wires wherever adjacent bars or deformed wires are cut off or bent in tension regions. In Figure CC-~~6.1-7~~6.1-8, the peak stress points remaining in continuing reinforcement after some of the reinforcement has been cut off are indicated by the notation "critical section". If reinforcement is terminated as short as the moment diagrams allow, these stresses become the full design stress which requires a full embedment length as indicated. This extension may exceed the length required for flexure.

6.1.10.1.5 Evidence of reduced shear strength and loss of ductility when reinforcement is cut off in a tension zone has been reported in Ferguson and Matloob (1959). As a result, the Code does not permit flexural reinforcement to be terminated in a tension zone unless special conditions are satisfied. Flexure cracks tend to open early wherever any reinforcement is terminated in a tension zone. If the stress in the continuing reinforcement and the shear strength are each near their limiting values, diagonal tension cracking tends to develop prematurely from these flexure cracks. Diagonal cracks are less likely to form where shear stress is low. A lower steel stress reduces the probability of such diagonal cracking.

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(i) three-fourths the allowable shear at the section considered, when using Chapter 8; or,

(ii) three-fourths the design strength in shear at the section considered, when using Chapter 9 or Chapter 11.

6.1.10.1.6 Anchorage complying with Section 6.1.6.1 or 6.1.6.2 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.

6.1.10.2 *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 the yield strength of the reinforcement in tension.

6.1.10.3 *Development of negative moment reinforcement*

6.1.10.3.1 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 6.1.6.

6.1.10.3.2 At least one-third of the total reinforcement provided for moment at a support shall extend beyond the point of inflection not less than the greatest of the following: d, $12d_b$, and one-sixteenth of the clear span.

6.2 — Metal accessories

6.2.1 *Protection of metal accessories*

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. Ties, anchors, and inserts shall be mill galvanized, hot-dip galvanized, or stainless steel for all other cases. Anchor bolts, and steel plates and bars, not exposed to earth, weather, nor exposed to a mean relative humidity exceeding 75 percent, need not be coated.

COMMENTARY

6.1.10.1.6 In corbels, deep flexural members, variable-depth arches, members where the tension reinforcement is not parallel with the compression face, or other instances where the steel stress, f_s, in flexural reinforcement does not vary linearly in proportion to the moment, special means of analysis should be used to determine the peak stress for proper development of the flexural reinforcement.

6.1.10.2 *Development of positive moment reinforcement* — When a flexural member is part of the lateral-force-resisting system, loads greater than those anticipated in design may cause reversal of moment at supports. As a consequence, some positive reinforcement is required to be anchored into the support. This anchorage assures ductility of response in the event of serious overstress, such as from blast or earthquake. The use of more reinforcement at lower stresses is not sufficient. The full anchorage requirement need not be satisfied for reinforcement exceeding 25 percent of the total that is provided at the support.

6.1.10.3 *Development of negative moment reinforcement* — Negative reinforcement must be properly anchored beyond the support faces by extending the reinforcement ℓ_d into the support or by anchoring of the reinforcement with a standard hook or suitable mechanical device.

Section 6.1.10.3.2 provides for possible shifting of the moment diagram at a point of inflection, as discussed under Commentary Section 6.1.10.1.3. This requirement may exceed that of Section 6.1.10.1.3 and the more restrictive governs. In lieu of providing the development lengths and bar extensions shown in Figure CC- 6.1-8, the reinforcing may be made continuous with an appropriate splice.

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6.2 — Metal accessories

6.2.1 *Protection of metal accessories*

Corrosion resistance requirements are included because masonry cover varies considerably for these items. The exception for anchor bolts is based on current industry practice.

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6.3 — Anchor bolts

Headed and bent-bar anchor bolts shall conform to the provisions of Sections 6.3.1 through 6.3.8.

6.3.1 *Placement*

Headed and bent-bar anchor bolts shall be embedded in grout. Anchor bolts of ¼ in. (6.4 mm) diameter are permitted to be placed in mortar bed joints that are at least ½ in. (12.7 mm) in thickness and, for purposes of application of the provisions of Sections 6.3, 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 ¼ in. (6.4 mm) of fine grout between the bolts and the masonry unit or ½ in. (12.7 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 ¼ in. (6.4 mm) of fine grout between the head or bent leg of each bolt and the masonry unit or ½ in. (12.7 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 1 in. (25.4 mm).

COMMENTARY

6.3 — Anchor bolts

These design values apply only to the specific types of bolts mentioned. These bolts are readily available and are depicted in Figure CC-6.3-1.

6.3.1 *Placement*

Most tests on anchor bolts in masonry have been performed on anchor bolts embedded in grout. Placement limits for anchor bolts are based on successful construction practice over many years. The limits are intended to facilitate the flow of grout between bolts and between bolts and the masonry unit.

Research at Portland State University (Rad et al (1998)) and at Washington State University (Tubbs et al (2000)) has established that there is no difference in the performance of an anchor bolt installed through a tight-fitting hole in the face shell of a grouted hollow masonry unit and in an over-sized hole in the face shell of a grouted hollow masonry unit. Therefore, the clear distance requirement for grout to surround an anchor bolt is not needed where the bolt passes through the face shell. See Figure CC-6.3-2.

Quality assurance/quality control (QA/QC) procedures should ensure that there is sufficient clearance around the bolts prior to grout placement. These procedures should also require observation during grout placement to ensure that grout completely surrounds the bolts, as required by Table 4 in TMS 602.

COMMENTARY

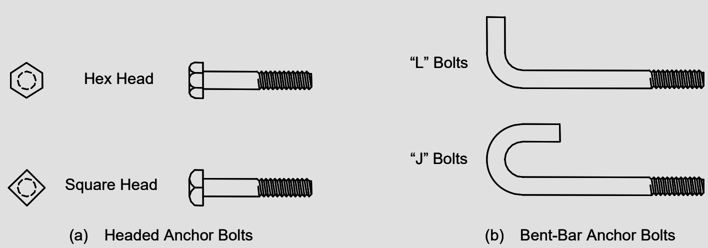

Figure CC-6.3-1 — Anchor bolts

COMMENTARY

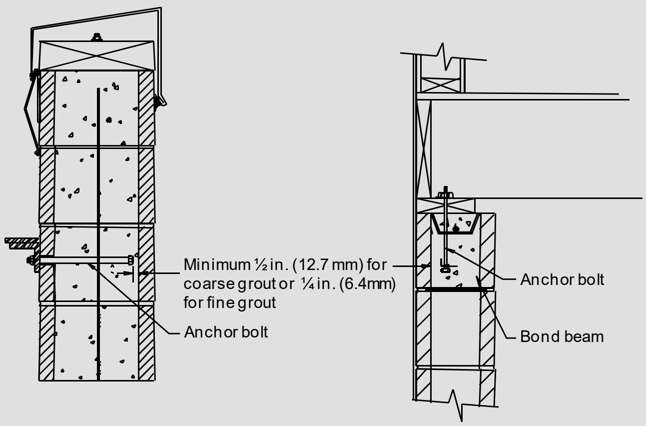

Figure CC-6.3-2 — Anchor bolt clearance requirements for headed anchor bolts – bent-bars are similar

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6.3.2 *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 Equation 6-5.

$$A_{pt} = \pi \ell_b^2 \quad \text{(Equation 6-5)}$$

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 Equation 6-5. Where the projected areas of anchor bolts overlap, the value of A_{pt} calculated using Equation 6-5 shall be adjusted so that no portion of masonry is included more than once.

COMMENTARY

6.3.2 *Projected area for axial tension*

Results of tests (Brown and Whitlock (1983); Allen et al (2000)) on headed anchor bolts in tension showed that anchor bolts often failed by breakout of a conically shaped section of masonry. The area, A_{pt}, is the projected area of the assumed failure cone. The cone originates at the compression bearing point of the embedment and radiates at 45 degrees in the direction of the pull (See Figure CC-6.3-3). Other modes of tensile failure are possible. These modes include pullout (straightening of J- or L-bolts) and yield / fracture of the anchor steel.

When anchor bolts are closely spaced, stresses within the masonry begin to become additive, as shown in Figure CC-6.3-4. The Code requires that when projected areas of anchor bolts overlap, an adjustment be made so that the masonry is not overloaded. When the projected areas of two or more anchors overlap, the anchors with overlapping projected areas should be treated as an anchor group. The projected areas of the anchors in the group are summed, this area is adjusted for overlapping areas, and the capacity of the anchor group is calculated using the adjusted area in place of A_{pt}. See Figure CC-6.3-5 for examples of calculating adjusted values of A_{pt}. The equations given in Figure CC-6.3-5 are valid only when the projected areas of the bolts overlap.

COMMENTARY

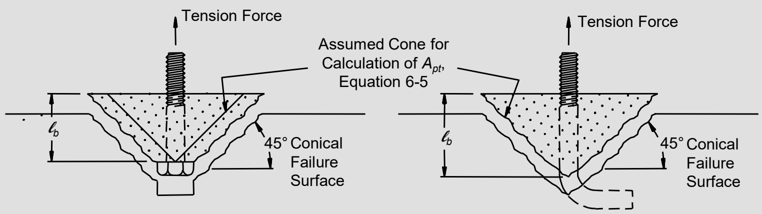


Figure CC-6.3-3 — Anchor bolt tensile breakout cone

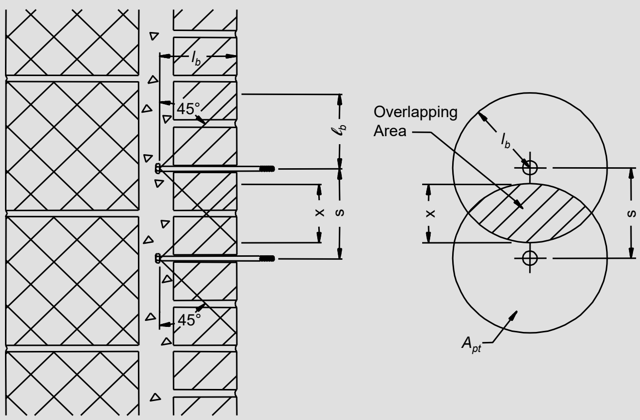
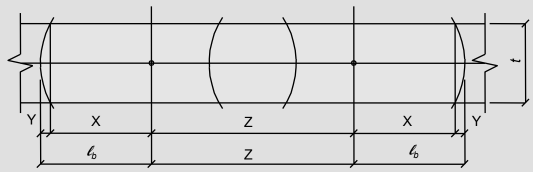


Figure CC-6.3-4 — Overlapping anchor bolt breakout cones

COMMENTARY

A_{pt} at Top of Wall for Uplift



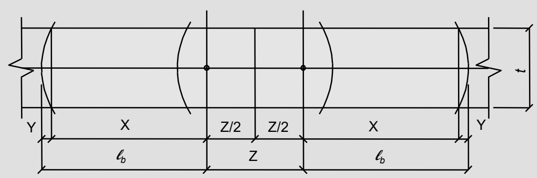
For $\ell_b \le z \le 2\mathrm{X}$

$$\mathrm{X} = \frac{1}{2}\sqrt{4\left(\ell_b\right)^2 - t^2}$$

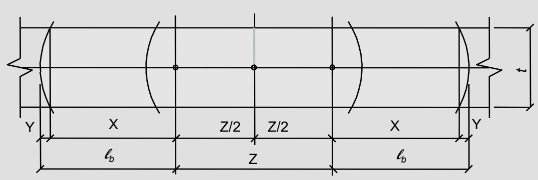
$$\mathrm{Y} = l_b - \mathrm{X} = l_b - \frac{1}{2}\sqrt{4\left(\ell_b\right)^2 - t^2}$$

$$\therefore A_{pt} = \left(2X + \mathrm{Z}\right)t + \ell_b^2\left(\frac{\pi\theta}{180} - \sin\theta\right) \quad \text{where } \theta = 2\arcsin\left(\frac{t/2}{\ell_b}\right) \text{ in degrees}$$

For $0 \le z \le \ell_b$



$$\therefore A_{pt} = \left(2X + \mathrm{Z}\right)t + \ell_b^2\left(\frac{\pi\theta}{180} - \sin\theta\right) \quad \text{where } \theta = 2\arcsin\left(\frac{t/2}{\ell_b}\right) \text{ in degrees}$$



$$\therefore A_{pt} = \left(2X + \mathrm{Z}\right)t + \ell_b^2\left(\frac{\pi\theta}{180} - \sin\theta\right) \quad \text{where } \theta = 2\arcsin\left(\frac{t/2}{\ell_b}\right) \text{ in degrees}$$

Figure CC-6.3-5 — Calculation of Adjusted Values of A_{pt} (Plan Views)

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6.3.3 *Projected area for shear*

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

$$A_{pv} = \frac{\pi \ell_{be}^2}{2} \qquad \text{(Equation 6-6)}$$

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 Equation 6-6. Where the projected areas of anchor bolts overlap, the value of A_{pv} calculated using Equation 6-6 shall be adjusted so that no portion of masonry is included more than once.

6.3.4 *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.

COMMENTARY

6.3.3 *Projected area for shear*

Results of tests (Brown and Whitlock (1983); Allen et al (2000)) on anchor bolts in shear showed that anchor bolts often failed by breakout of a conically shaped section of masonry. The area A_{pv} is the projected area of the assumed failure cone. The cone originates at the compression bearing point of the embedment and radiates at 45 degrees in the direction of the shear force towards the free edge of the masonry, thereby accounting for bolt edge distance (See Figure CC-6.3-6). Edge distance is considered in the calculation of the projected area for shear, A_{pv} (Equation 6-6). No minimum edge distance is provided for the placement of anchor bolts. Placement of all anchors, including anchor bolts, must meet the minimum thickness of grout between the anchor and masonry units given in Section 6.3.1. Pryout (See Figure CC-6.3-7), masonry crushing, and yielding / fracture of the anchor steel are other possible failure modes.

When the projected areas of two or more anchors overlap, the shear design of these anchors should follow the same procedure as for the tension design of overlapping anchors. See Commentary Section 6.3.2.

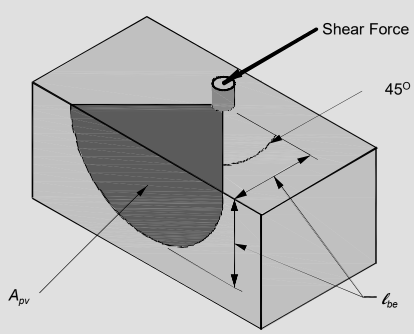

Figure CC-6.3-6 — Anchor bolt shear breakout

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6.3.5 *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.

COMMENTARY

6.3.5 *Effective embedment length for bent-bar anchor bolts*

Tests (Brown and Whitlock (1983)) have shown that the pullout strength of bent-bar anchor bolts correlated best with a reduced embedment length. This may be explained with reference to Figure CC-6.3-8. Due to the radius of the bend, stresses are concentrated at a point less than the full embedment length.

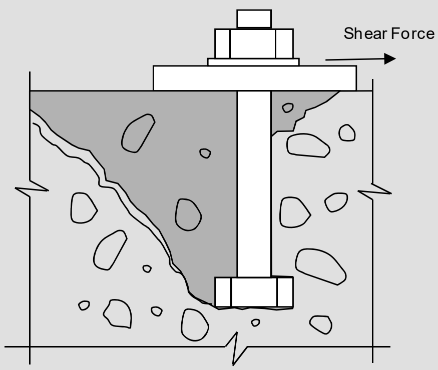

Figure CC-6.3-7 — Anchor bolt shear pryout

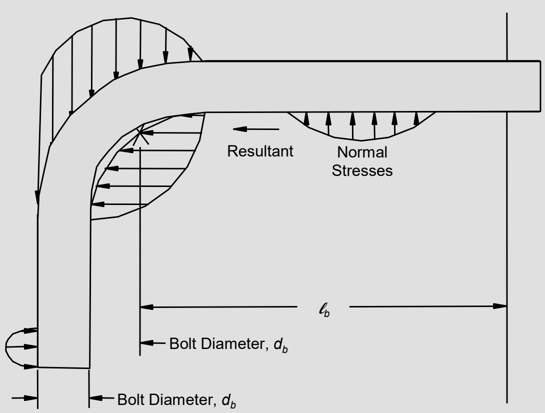

Figure CC-6.3-8 — Stress distribution on bent-bar anchor bars

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6.3.6 *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 2 in. (50.8 mm).

6.3.7 *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.

6.3.8 *Effective cross-sectional area of threaded anchor bolts*

The effective cross-sectional area of a threaded anchor bolt shall be determined from Equation 6-7.

$$A_b = \frac{\pi}{4}\left(d_o - \frac{0.9743}{n_t}\right)^2 \qquad \text{(Equation 6-7)}$$

COMMENTARY

6.3.6 *Minimum permissible effective embedment length*

The minimum embedment length requirement is considered a practical minimum based on typical construction methods for embedding anchor bolts in masonry. The validity of Code equations for shear and tension capacities of anchor bolts have not been verified by testing of anchor bolts with embedment lengths less than four bolt diameters.

6.3.8 *Effective cross-sectional area of threaded anchor bolts*

The effective cross-sectional area of a threaded anchor bolt, Equation 6-7, including the effect of threads, is obtained from ASME B1.1 (2003). The value of A_b for common anchor bolt sizes used in masonry is given in Table CC-6.3.1.

Table CC-6.3.1: Anchor bolt effective cross-sectional area

Bolt size - threads per inch	A_b (in.^{2})
1/2 – 13	0.142
5/8 – 11	0.226
3/4 – 10	0.334
7/8 – 9	0.462
1– 8	0.606

CHAPTER 7
SEISMIC DESIGN REQUIREMENTS

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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.

COMMENTARY

7.1 — Scope

The requirements in this section have been devised to improve performance of masonry construction when subjected to earthquake loads. Minimum seismic loading requirements are drawn from the legally adopted building code. In the event that the legally adopted building code does not contain appropriate criteria for the determination of seismic forces, the Code requires the use of ASCE/SEI 7, which represented the state-of-the-art in seismic design at the time these requirements were developed. The seismic design provisions of this section may not be compatible with every edition of every building code that could be used in conjunction with these requirements. As with other aspects of structural design, the designer should understand the implications and limits of combining the minimum loading requirements of other documents with the resistance provisions of this Code.

Seismic design is not optional regardless of the assigned Seismic Design Category, the absolute value of the design seismic loads, or the relative difference between the design seismic loads and other design lateral forces such as wind. Unlike other design loads, seismic design of reinforced masonry elements permits inelastic response of the system, which in turn reduces the seismic design load. This reduction in load presumes an inherent level of inelastic ductility that may not otherwise be present if seismic design was neglected. When nonlinear response is assumed by reducing the seismic loading by an R factor greater than 1.5, the resulting seismic design load may be less than other loading conditions that assume a linear elastic model of the system. This is often misinterpreted by some to mean that the seismic loads do not ‘control’ the design and can be neglected. For the masonry system to be capable of achieving the ductility-related lower seismic loads, however, the minimum seismic design and detailing requirements of this section must be met.

The seismic design requirements are presented in a cumulative format. Thus, the provisions for Seismic Design Categories E and F include provisions for Seismic Design Category D, which include provisions for Seismic Design Category C, and so on.

This section does not apply to the design or detailing of masonry veneers or glass unit masonry systems. Seismic requirements for masonry veneers are provided in Chapter 13, Veneers. 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.

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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 the legally adopted building code. When the legally adopted building code does not prescribe minimum load path connection requirements and anchorage design forces, the requirements of ASCE/SEI 7 shall be used.

7.2.4 *Drift limits* — Under loading combinations that include earthquake, masonry structures shall be designed so the ~~calculated~~ design story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from the legally adopted building code. When the legally adopted building code does not prescribe allowable story drifts, structures shall be designed so the ~~calculated~~ design story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from ASCE/SEI 7.

It shall be permitted to assume that the following shear wall types comply with the story drift limits of ASCE/SEI 7: ordinary plain, detailed plain, ordinary reinforced, intermediate reinforced, ordinary plain AAC masonry shear walls, and detailed plain AAC masonry shear walls.

COMMENTARY

7.2 — General analysis

The designer is permitted to use any of the structural design methods presented in this Code to design to resist seismic loads. There are, however, limitations on some of the design methods and systems based upon the structure's assigned Seismic Design Category. For instance, unreinforced masonry is not permitted to be used as part of the lateral force-resisting system in structures assigned to Seismic Design Categories C, D, E, and F.

7.2.1 *Element interaction* — Even if a nonstructural element is not part of the seismic-force-resisting system, it is possible for it to influence the structural response of the system during a seismic event. This may be particularly apparent due to the interaction of structural and nonstructural elements at displacements larger than those determined by linear elastic analysis.

7.2.2 *Load path* — This section clarifies load path requirements and alerts the designer that the base of the structure as defined in analysis may not necessarily correspond to the ground level.

7.2.3 *Anchorage design* — Experience has demonstrated that one of the chief causes of failure of masonry construction during earthquakes is inadequate anchorage of masonry walls to floors and roofs. For this reason, an arbitrary minimum anchorage based upon previously established practice has been set as noted in the referenced documents. When anchorage is between masonry walls and wood framed floors or roofs, the designer should avoid the use of wood ledgers in cross-grain bending.

7.2.4 *Drift limits* — Excessive deformation, particularly resulting from inelastic displacements, can potentially result in instability of the seismic-force-resisting system. This section provides procedures for the limitation of story drift. The term "drift" has two connotations:

1. "Story drift" is the maximum calculated lateral displacement within a story (the calculated displacement of one level relative to the level below caused by the effects of design seismic loads). In this Code, notation using an upper case delta (Δ) is used to indicate relative story displacements.
2. The calculated lateral displacement or deflection due to design seismic loads is the absolute displacement of any point in the structure relative to the base. This is not "story drift" and is not to be used for drift control or stability considerations because it may give a false impression of the effects in critical stories. However, it is important when considering seismic separation requirements and is used in determining rotation demands on cantilevered walls and limit mechanisms. In this Code, notation using a lower case delta (δ) is used to indicate displacements relative to the base.

Overall or total drift is the lateral displacement of the top of a building relative to the base. The overall drift ratio is the

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total drift divided by the building height. Story drift is the lateral displacement of one story relative to an adjacent story. The story drift ratio is the story drift divided by the corresponding story height. The overall drift ratio is usually an indication of moments in a structure and is also related to seismic separation demands. The story drift ratio is an indication of local seismic deformation, which relates to seismic separation demands within a story. The maximum story drift ratio could exceed the overall drift ratio.

There are many reasons for controlling drift in seismic design:

(a) To control the inelastic strain within the affected elements. Although the relationship between lateral drift and maximum nonlinear strain is imprecise, so is the current state of knowledge of what strain limitations should be.

(b) Under small lateral deformations, secondary stresses are normally within tolerable limits. However, larger deformations with heavy vertical loads can lead to significant secondary moments from P-delta effects in the design. The drift limits indirectly provide upper bounds for these effects.

(c) Buildings subjected to earthquakes need drift control to restrict damage to partitions, shaft and stair enclosures, glass, and other fragile nonstructural elements and, more importantly, to minimize differential movement demands on the seismic-force-resisting elements.

The designer must keep in mind that the allowable drift limits, Δ_a, correspond to story drifts and, therefore, are applicable to each story. They must not be exceeded in any story even though the drift in other stories may be well below the limit.

Although the provisions of this Code do not give equations for calculating building separations, ASCE/SEI 7 can be used to determine the distance ~~should~~ that would be sufficient to avoid damaging contact should such requirements be absent from the legally adopted building code. ~~under total calculated deflection for the design loading in order to avoid interference and possible destructive hammering between buildings. The distance should be equal to the total of the lateral deflections of the two units assumed deflecting toward each other (this involves increasing the separation with height). If the effects of hammering can be shown not to be detrimental, these distances may be reduced.~~ For very rigid shear wall structures with rigid diaphragms whose lateral deflections are difficult to estimate, older code requirements for structural separations of at least 1 in. (25.4 mm) plus ½ in. (12.7 mm) for each 10 ft (3.1 m) of height above 20 ft (6.1 m) could be used as a guide.

Ordinary plain, detailed plain, ordinary reinforced, intermediate reinforced, ordinary plain AAC, and detailed plain AAC masonry shear walls are inherently designed to have relatively low inelastic deformations under seismic loads. As such, the Committee felt that requiring designers to check

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story drifts for these systems of low and moderate ductility was not ~~exceeded~~warranted.

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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. Isolation joints and connectors shall be designed to accommodate the design story drift.

Exception: Isolation is not required if a deformation compatibility analysis demonstrates that the non-participating element can accommodate the inelastic story drift~~displacement~~, ~~$C_d\delta_{ne}$~~Δ, of the structure in a manner complying with the requirements of this C~~c~~ode. Elements supporting gravity loads in addition their self-weight shall be evaluated for gravity load combinations of (1.2D + 1.0L + ~~0.2~~0.15S) or 0.9D, whichever is critical, acting simultaneously with the inelastic displacement and shall have a ductility compatible with the ductility of the ~~lateral~~ seismic-force-resisting system. The influence of any non-isolated nonparticipating elements on the ~~lateral~~ seismic-force-resisting system shall be considered in design in accordance with Section 4.1.6 of this C~~c~~ode.

COMMENTARY

7.3 — Element classification

Classifying masonry elements as either participating or nonparticipating in the seismic-force-resisting system is largely a function of design intent. Participating elements are those that are designed and detailed to actively resist seismic forces, including such elements as shear walls, columns, pilasters, beams, and coupling elements. Nonparticipating elements can be any masonry assembly, but are not designed to collect and resist earthquake loads from other portions of the structure.

7.3.1 *Nonparticipating elements* — With regards to the exception,~~,~~ non-isolated, nonparticipating elements can influence a structure's strength and stiffness, and as a result the distribution of lateral loads and building irregularities. ~~The influence of any n~~Non-isolated nonparticipating elements can inadvertently have significant effects on the performance of a structural system and are to be considered in accordance with the code. This should also be considered ~~in design~~ in accordance with ~~Section 4.1.6 of this code, and~~ other applicable provisions such as the modelling criteria of ASCE/SEI 7. Where partial height non-participating elements are constructed tight to building columns, this should include the consideration of short column effects.~~, t~~

The deformation compatibility analysis may consider the effect of cracking on element stiffness. Elements that are detailed to achieve ductile behavior may also develop plastic mechanisms. For example, elements detailed in accordance with the provisions for special reinforced masonry shear walls may be able to accommodate displacements through the development of plastic hinges. For such elements, Appendix C may be used to provide guidance on the determination of hinge rotation capacity. In addition to these provisions, other applicable provisions, such as the deformation limit and deformation compatibility provisions of ASCE/SEI 7 should be considered in design.

When the lateral-force-resisting system consists of masonry shear walls, a nonparticipating element can achieve a ductility compatible with the ductility of the lateral-force resisting system by detailing the nonparticipating element to the same minimum requirements as the shear walls. For lateral-force-resisting systems constructed of other materials, a nonparticipating element can achieve a ductility compatible with the ductility of the lateral-force-resisting system ~~can be achieved~~ by detailing the nonparticipating element in accordance with the requirements for a masonry shear wall with an R value not less than that of the lateral-force-resisting system. If the lateral-force-resisting system has an R value in excess of that achievable for a special reinforced masonry shear wall, the non-participating element will not qualify for the exception.

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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, 7.3.2.9, 7.3.2.10, or 7.3.2.11.

7.3.2 *Participating elements* — A seismic-force-resisting system must be defined for every structure. Most masonry buildings use masonry shear walls to serve as the seismic-force-resisting system, although other systems are sometimes used (such as concrete or steel frames with masonry infill). Such shear walls must be designed by the engineered methods in Part 3.

Eleven shear wall types are defined in this section. Depending upon the masonry material and detailing method, inelastic response capacity and energy dissipation may vary for each wall type. Eight of these shear wall types are assigned system design parameters per ASCE/SEI 7, which include response modification factors, R, overstrength factors, Ω_0 and deflection amplification factors, C_d, based on their expected performance and ductility. Section 12.2.1 of ASCE/SEI 7 permits wall types not addressed by that standard provided that analytical and test data are submitted to authorities having jurisdiction. Certain shear wall types are permitted in each seismic design category, however ~~unreinforced~~ plain shear wall types are not permitted in regions of intermediate and high seismic risk. Requirements for each of the eleven shear wall types, as given in Section 7.3.2, are summarized in Table CC-7.3.2-1.

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7.3.2.1 *Ordinary plain masonry shear walls* — Design of ordinary plain masonry shear walls shall comply with the requirements of Section 8.2 or Section 9.2.

7.3.2.1 *Ordinary plain masonry shear walls* — These shear walls are permitted to be used only in Seismic Design Categories A and B. Plain masonry walls are designed as unreinforced masonry, although they may in fact contain reinforcement.

Table CC-7.3.2-1: Requirements for Masonry Shear Walls Based on Shear Wall Designation

Shear Wall Designation	Design Methods	Reinforcement Requirements	Permitted In
Ordinary Plain Masonry Shear Walls	Section 8.2 or Section 9.2	None	SDC A and B
Detailed Plain Masonry Shear Walls	Section 8.2 or Section 9.2	Section 7.3.2.2.1	SDC A and B
Ordinary Reinforced Masonry Shear Walls	Section 8.3 or Section 9.3	Section 7.3.2.2.1	SDC A, B, and C
Intermediate Reinforced Masonry Shear Walls	Section 8.3 or Section 9.3	Section 7.3.2.4	SDC A, B, and C
Special Reinforced Masonry Shear Walls	Section 8.3 or Section 9.3	Section 7.3.2.5	SDC A, B, C, D, E, and F
Ordinary Plain AAC Masonry Shear Walls	Section 11.2	Section 7.3.2.6.1	SDC A and B
Detailed Plain AAC Masonry Shear Walls	Section 11.2	Section 7.3.2.7.1	SDC A and B
Ordinary Reinforced AAC Masonry Shear Walls	Section 11.3	Section 7.3.2.8	SDC A, B, C, D, E, and F
Ordinary Plain Prestressed Masonry Shear Walls	Chapter 10	None	SDC A and B
Intermediate Reinforced Prestressed Masonry Shear Walls	Chapter 10	Section 7.3.2.10	SDC A, B, and C
Special Reinforced Prestressed Masonry Shear Walls	Chapter 10	Section 7.3.2.11	SDC A, B, C, D, E, and F

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7.3.2.2 *Detailed plain masonry shear walls* — Design of detailed plain masonry shear walls shall comply with the requirements of Section 8.2 or Section 9.2, and shall comply with the requirements of Section 7.3.2.2.1.

7.3.2.2.1 *Minimum reinforcement requirements* — Vertical reinforcement of at least 0.2 in.^{2} (129 mm^2) in cross-sectional area shall be provided at corners, within 16 in. (406 mm) of each side of openings, within 8 in. (203 mm) of each side of movement joints, within 8 in. (203 mm) of the ends of walls, and at a maximum spacing of 120 in. (3048 mm) on center.

Vertical reinforcement adjacent to openings need not be provided for openings smaller than 16 in. (406 mm), unless the distributed reinforcement is interrupted by such openings.

Horizontal reinforcement shall consist of at least two longitudinal wires of W1.7 (MW11) joint reinforcement spaced not more than 16 in. (406 mm) on center, or at least 0.2 in.^{2} (129 mm^2) in cross-sectional area of bond beam reinforcement spaced not more than 120 in. (3048 mm) on center. Horizontal reinforcement shall also be provided: at the

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7.3.2.2 *Detailed plain masonry shear walls* — These shear walls are designed as unreinforced masonry in accordance with the sections noted, but contain minimum reinforcement in the horizontal and vertical directions. Walls that are designed as unreinforced, but that contain minimum prescriptive reinforcement, have more favorable seismic design parameters, including higher response modification coefficients, R, than ordinary plain masonry shear walls.

7.3.2.2.1 *Minimum reinforcement requirements* — The provisions of this section require a judgment-based minimum amount of reinforcement to be included in reinforced masonry wall construction. Tests reported in Gulkan et al (1979) have confirmed that masonry construction, reinforced as indicated, performs adequately considering the highest Seismic Design Category permitted for this shear wall type. This minimum required reinforcement may also be used to resist design loads.

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bottom and top of wall openings and shall extend at least 24 in. (610 mm) but not less than 40 reinforcement diameters past the opening; continuously at structurally connected roof and floor levels; and within 16 in. (406 mm) of the top of walls.

Horizontal reinforcement adjacent to openings need not be provided for openings smaller than 16 in. (406 mm), unless the distributed reinforcement is interrupted by such openings.

7.3.2.3 *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 7.3.2.2.1.

7.3.2.4 *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 7.3.2.2.1, except that the spacing of vertical reinforcement shall not exceed 48 in. (1219 mm).

7.3.2.5 *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 7.3.2.2.1 and the following:

(a) In-plane flexural reinforcement shall be deformed reinforcing bars.

(b) 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 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry not laid in running bond.

(c) The maximum spacing of horizontal reinforcement shall not exceed 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry not laid in running bond.

(d) The maximum spacing of horizontal shear reinforcement ~~required to resist in-plane shear~~ shall be the smaller of one-third the length of the shear wall and one-third the height of the shear wall. Horizontal shear reinforcement ~~required to resist in-plane shear~~ shall be uniformly distributed.

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7.3.2.3 *Ordinary reinforced masonry shear walls* — These shear walls are required to meet minimum requirements for reinforced masonry as noted in the referenced sections. Because they contain reinforcement, these walls can generally accommodate larger deformations and exhibit higher capacities than similarly configured plain masonry walls. Hence, they are permitted in both areas of low and moderate seismic risk. Additionally, these walls have more favorable seismic design parameters, including higher response modification factors, *R*, than plain masonry shear walls. To provide the minimum level of assumed inelastic ductility, however, minimum reinforcement is required as noted in Section 7.3.2.2.1.

7.3.2.4 *Intermediate reinforced masonry shear walls* — These shear walls are designed as reinforced masonry as noted in the referenced sections, and are also required to contain a minimum amount of prescriptive reinforcement. Because they contain reinforcement, their seismic performance is better than that of plain masonry shear walls, and they are accordingly permitted in both areas of low and moderate seismic risk. Additionally, these walls have more favorable seismic design parameters including higher response modification factors, *R*, than plain masonry shear walls and ordinary reinforced masonry shear walls.

7.3.2.5 *Special reinforced masonry shear walls* — These shear walls are designed as reinforced masonry as noted in the referenced sections and are also required to meet restrictive reinforcement and material requirements. Accordingly, they are permitted to be used as part of the seismic-force-resisting system in any Seismic Design Category. Additionally, these walls have the most favorable seismic design parameters, including the highest response modification factor, *R*, of any of the masonry shear wall types.

(a) The reinforcing wire products – joint reinforcing, deformed wire and welded wire reinforcement – are cold worked and lack the ductility required for flexural reinforcement in special reinforced masonry shear walls.

Subsections (c), (d), and ~~through~~ (f) stipulate a minimum level of in-plane shear reinforcement to improve ductility.

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(e) Joint reinforcement and deformed wire placed in mortar used as shear reinforcement~~required to resist in-plane shear~~ shall be a single piece without splices for the length of the wall used for shear design, d_v.

(~~d~~e) At this time, splicing of joint reinforcing and deformed wire placed in mortar is not permitted as research has not been done on the performance of lap splices of reinforcement placed in mortar under cyclic loads, and in mortar joints that may be cracked due to in-plane or out-of-plane loads. Where a wall is divided into two or more segments by movement joints, each segment will have its own length, d_v, and the joint reinforcing or deformed wires can be terminated in accordance with (g) on either side of the joint.

Joint reinforcing is also subject to the minimum reinforcement requirements based on Seismic Design Category, see Sections 7.4.1.2.1 and 7.4.3.2.6.

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(f) ~~The vertical reinforcement ratio shall be at least one-third of the horizontal reinforcement ratio required to resist in-plane shear.~~ The sum of the horizontal reinforcement ratio and vertical reinforcement ratio shall be at least 0.002. Reinforcement ratios shall be based on the gross cross-sectional area of the wall, using specified dimensions and shall be not less than the following:~~.~~

1. For masonry laid in running bond, the minimum reinforcement ratio in each direction shall be at least 0.0007.
2. For masonry not laid in running bond, the minimum vertical reinforcement ratio shall be at least 0.0007. The minimum horizontal reinforcement ratio shall be at least 0.0015.

Reinforcement used for compliance with these provisions shall be uniformly distributed.

(f) ~~In previous editions of the Code,~~ Prior to the 2022 edition of this Code, this section included a requirement for a minimum amount of vertical reinforcement based on the amount of horizontal shear reinforcement ~~required to resist shear~~. This requirement for a minimum amount of vertical reinforcement was redundant with provisions applicable to all reinforced masonry shear wall designs in Chapters 8 and 9 and has been removed from this section.~~The horizontal reinforcement ratio required to resist in-plane shear is determined by dividing the area of horizontal steel required to resist in-plane shear by the gross cross-sectional area of the wall in the vertical plane.~~

The minimum amount of wall reinforcement for special reinforced masonry shear walls has been a long-standing, standard empirical requirement in areas of high seismic loading. It is expressed as a percentage of gross cross-sectional area of the wall. It is intended to improve the ductile behavior of the wall under earthquake loading and assist in crack control.

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(g) Joint reinforcement used as shear reinforcement shall be anchored in accordance with Section 6.1.8.~~1.3~~2.1 (a) or (c) when two longitudinal wires are used and Section 6.1.8.~~1.3~~2.2 when four longitudinal wires are used. Deformed wire embedded in mortar and used as

(g) Option (a) in Section 6.1.8.2 and Option (b) in Section 6.1.8.2~~1.3~~.1 ~~is~~are excluded from use in special reinforced masonry shear walls due to lack of testing. Section 6.1.8.1 also addresses the anchorage of reinforcing bars

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shear reinforcement shall be anchored in accordance with Section 6.1.8.1 (b).

(h) Mechanical splices in flexural reinforcement in plastic hinge zones shall meet the requirements of Section 6.1.7.2.1 and develop the specified tensile strength of the spliced bar.

(i) When the ratio of $V/F_{vm}A_{nv}$ for masonry designed in accordance with Chapter 8 or when the ratio $V_u/\phi V_{nm}$ for masonry designed in accordance with Chapter 9, 10, or 11 exceeds 0.40, the termination of horizontal reinforcement embedded in grout shall meet one of the following:

1. Except at wall intersections, the ends of horizontal reinforcement shall be bent around the edge vertical reinforcement with a 180-degree standard hook.

2. At wall intersections, horizontal reinforcement shall be bent around the edge vertical reinforcement with a 90-degree standard hook and shall extend horizontally into the intersecting wall a minimum distance at least equal to the development length.

(j~~i~~) 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.

(k~~j~~) Welded splices in reinforcement shall not be permitted in plastic hinge zones.

7.3.2.5.1 *Shear capacity design*

7.3.2.5.1.1 When designing special reinforced masonry shear walls in accordance with Section 8.3.5, the calculated shear stress, f_v, or diagonal tension stress resulting from in-plane seismic forces shall be increased by a factor of 2.0.

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and deformed wires placed in grout used as shear reinforcement in walls.

(h) In a structure undergoing inelastic deformations during an earthquake, the tensile stresses in flexural reinforcement in plastic hinge zones may approach the tensile strength of the reinforcement. This requirement is intended to avoid a splice failure in such reinforcement. In a perforated or coupled shear wall, plastic hinge zones may form at locations other than at the base of the wall, such as at the interfaces between horizontal and vertical wall segments. Mechanical splices in these regions are required to develop the specified tensile strength of the bar.

For the purpose of this section, the plastic hinge zone may be assumed to extend at least half of the member depth from the plane where yielding is expected to initiate.

(i) Research (Seif Eldin (2017)) has shown an increase in the ductility of masonry piers where the horizontal reinforcement is hooked around the edge vertical bar. When the demand-to-resistance ratio is less than 40%, inelastic response is generally expected to be associated with low ductility demands where the benefit of prescriptive hooks for shear reinforcement is marginal.

(k~~j~~) Welding can adversely affect the ductility of the reinforcement, and is thus prohibited in plastic hinge zones. See commentary for item (~~f~~h) for additional discussion of plastic hinge zones.

7.3.2.5.1 *Shear capacity design* — Sections 7.3.2.5.1.1 and 7.3.2.5.1.2 attempt to limit shear failures prior to nonlinear flexural behavior – or if one prefers – increase element ductility.

7.3.2.5.1.1 The 2.0 load factor for special reinforced masonry shear walls that are part of the seismic-force~~-~~resisting system designed by allowable stress design procedures is applied only to in-plane shear forces. It is not intended to be used for the design of in-plane overturning moments or out-of-plane overturning moments or shear. Increasing the design seismic load is intended to make the flexure mode of failure more dominant, resulting in more ductile performance. The 2.0 multiplier should not

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7.3.2.5.1.2 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 design shear strength, ϕV_n, need not exceed 2.0 times required shear strength, V_u.

7.3.2.6 *Ordinary plain AAC masonry shear walls* — Design of ordinary plain AAC masonry shear walls shall comply with the requirements of Section 11.2 and Section 7.3.2.6.1.

7.3.2.6.1 *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 or deformed wires, having a total cross-sectional area of at least 0.4 in.^{2} (260 mm^2).

7.3.2.7 *Detailed plain AAC masonry shear walls* — Design of detailed plain AAC masonry shear walls shall comply with the requirements of Section 11.2 and Sections 7.3.2.6.1 and 7.3.2.7.1.

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be applied to V when calculating the M/Vd_v ratio, or for shear-friction design, or for determining compliance with 7.3.2.5 (i).

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7.3.2.5.1.2 The effect of axial load on increasing flexural strength, M_n, and thus, shear demand, as well as the effect of axial load on influencing shear capacity, V_n, should be considered.

The provisions of this Section only apply to the nominal shear strength, V_n, and do not apply to the nominal shear friction strength, V_{nf}, nor do they apply when determining compliance with 7.3.2.5 (i).

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7.3.2.6 *Ordinary plain AAC masonry shear walls* — These shear walls are philosophically similar in concept to ordinary plain masonry shear walls. As such, prescriptive mild reinforcement is not required, but may actually be present.

7.3.2.7 *Detailed plain AAC masonry shear walls* — Prescriptive seismic requirements for AAC masonry shear walls are less severe than for conventional masonry shear walls, and are counterbalanced by more restrictive Code requirements for bond beams and additional requirements for floor diaphragms, contained in evaluation service reports and other documents dealing with floor diaphragms of various materials. AAC masonry shear walls and a full-scale, two-story assemblage specimen with prescriptive reinforcement meeting the requirements of this section have performed satisfactorily under reversed cyclic loads representing seismic excitation (Varela et al (2006); Tanner et al (2005)). The maximum distance from the edge of an opening or end of a wall to the vertical reinforcement is set at 24 in. (610 mm) because the typical length of an AAC unit is 24 in. (610 mm).

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7.3.2.7.1 *Minimum reinforcement requirements* — Vertical reinforcement of at least 0.2 in.^{2} (129 mm^2) shall be provided within 24 in. (610 mm) of each side of openings, within 8 in. (203 mm) of movement joints, and within 24 in. (610 mm) of the ends of walls. Vertical reinforcement adjacent to openings need not be provided for openings smaller than 16 in. (406 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 24 in. (610 mm) but not less than 40 reinforcement diameters past the opening. Horizontal reinforcement adjacent to openings need not be provided for openings smaller than 16 in. (406 mm), unless the distributed reinforcement is interrupted by such openings.

7.3.2.8 *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 7.3.2.6.1 and 7.3.2.7.1.

7.3.2.8.1 *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 design shear strength, ϕV_n, need not exceed 2.0 times required shear strength, V_u.

7.3.2.9 *Ordinary plain prestressed masonry shear walls* — Design of ordinary plain prestressed masonry shear walls shall comply with the requirements of Chapter 10.

7.3.2.10 *Intermediate reinforced prestressed masonry shear walls* — Intermediate reinforced prestressed masonry shear walls shall comply with the requirements of Chapter 10, the reinforcement detailing requirements of Section 7.3.2.2.1, and the following:

(a) Reinforcement shall be provided in accordance with Sections 7.3.2.5(~~a~~b), 7.3.2.5(c), and 7.3.2.5(~~b~~d).

(b) The minimum area of horizontal reinforcement shall be $0.0007bd_v$.

(c) Shear walls subjected to load reversals shall be symmetrically reinforced.

(d) The nominal moment strength at any section along the shear wall shall not be less than one-fourth the maximum moment strength.

(e) The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Sections 7.3.2.2.1, 7.3.2.5(~~a~~b), 7.3.2.5(c), and 7.3.2.5(~~b~~d).

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7.3.2.9 *Ordinary plain prestressed masonry shear walls* — These shear walls are philosophically similar in concept to ordinary plain masonry shear walls. As such, prescriptive mild reinforcement is not required, but may actually be present. Seismic design factors provided for this type of prestressed masonry shear walls in ASCE/SEI 7~~-16~~ are in approximate agreement with the R and C_d factors recommended by Hassanli et al (2015) for ungrouted prestressed masonry shear walls.

7.3.2.10 *Intermediate reinforced prestressed masonry shear* walls — These shear walls are philosophically similar in concept to intermediate reinforced masonry shear walls. To provide the intended level of inelastic ductility, prescriptive ~~mild~~ reinforcement is required. Intermediate reinforced prestressed masonry shear walls should include ~~the detailing requirements from Section 7.3.2.4 and~~ the sectional ductility (a/d) requirement in Section 10.5.3.

ASCE/SEI 7, Tables 12.2-1 and 12.14-1 conservatively combine all prestressed masonry shear walls into one category for seismic coefficients and structural system limitations on seismic design categories and height. The design limitations included in those tables are representative of ordinary plain prestressed masonry shear walls. Given that an intermediate prestressed masonry shear wall can be partially grouted, Hassanli et al (2015) recommend R and C_d factors of 2½ and 2.9, respectively. To utilize the seismic design factors proposed by Hassanli et al (2015), the structure would have to be accepted under Section 1.3, Alternative design or method of construction.

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(f) Tendons shall be located in cells that are grouted the full height of the wall.

7.3.2.11 *Special reinforced prestressed masonry shear walls* — Special reinforced prestressed masonry shear walls shall comply with the requirements of Chapter 10, the reinforcement detailing requirements of Sections 7.3.2.2.1 and 7.3.2.10 and the following:

(a) The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Sections 7.3.2.2.1 and 7.3.2.10.

(b) Prestressing tendons shall consist of bars conforming to ASTM A722/A722M.

(c) All cells of the masonry wall shall be grouted.

(d) The requirements for special reinforced shear walls of Section 9.3.5.6 shall be met. Dead load axial forces shall include the effective prestress force, $A_{ps}f_{se}$.

(e) 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 design shear strength, ϕV_n, need not exceed 2.0 times required shear strength, V_u.

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7.3.2.11 *Special reinforced prestressed masonry shear walls* — These shear walls are philosophically similar in concept to special reinforced masonry shear walls. To provide the intended level of inelastic ductility, prescriptive ~~mild~~ reinforcement is required. Special reinforced prestressed masonry shear walls should include ~~the detailing requirements from Section 7.3.2.5 and~~ the sectional ductility (*a/d*) requirement in Section 10.5.3.

ASCE/SEI 7, Table 12.2-1 and ASCE/SEI 7, Table 12.14-1 conservatively combine all prestressed masonry shear walls into one category for seismic coefficients and structural system limitations on seismic design categories and height. The design limitations included in those tables are representative of ordinary plain prestressed masonry shear walls. Given that a special prestressed masonry shear wall must be fully grouted, Hassanli et al (2015) recommend R and C_d factors of 3 and 3½, respectively. To utilize the seismic design factors proposed by Hassanli et al (2015), the structure would have to be accepted under Section 1.3, Alternative design or method of construction.

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7.4 — Seismic Design Category requirements

The design of masonry elements shall comply with the requirements of Sections 7.4.1 through 7.4.5 based on the Seismic Design Category as defined in the legally adopted building code. When the legally adopted building code does not define Seismic Design Categories, the provisions of ASCE/SEI 7 shall be used.

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, 9, 10, 11, 12, 15, or Appendix D.

7.4.1.2 *Design of participating elements* — Participating masonry elements shall be designed to comply with the requirements of Chapter 8, 9, 10, 11, or 12. 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, 7.3.2.9, 7.3.2.10, or 7.3.2.11.

7.4.1.2.1 *Joint reinforcement used as shear reinforcement* — Horizontal joint reinforcement used as shear reinforcement in walls shall consist of at least two 3/16 in. (4.8 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 16 in. (406 mm).

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7.4 — Seismic Design Category requirements

Every structure is assigned to a Seismic Design Category (SDC) in accordance with the legally adopted building code or per the requirements of ASCE/SEI 7, whichever govern for the specific project under consideration.

7.4.1 *Seismic Design Category A requirements* — The general requirements of this Code provide for adequate performance of masonry construction assigned to Seismic Design Category A structures.

7.4.1.2.1 *Joint reinforcement used as shear reinforcement* — The quantities of joint reinforcement indicated are minimums and the designer should evaluate whether additional reinforcement is required to satisfy specific seismic conditions.

Studies of minimum shear reinforcement requirements (Schultz (1996); Baenziger and Porter (2018); Baenziger and Porter (2011); Porter and Baenziger (2007); Sveinsson et al (1985); Schultz and Hutchinson (2001)) have shown that when sufficient area, strength, and strain elongation properties of reinforcement are provided to resist the load transferred from the masonry after cracking, then the reinforcement does not rupture upon cracking of the masonry. Equivalent performance of shear walls with bond beams and shear walls with bed joint reinforcement under simulated seismic loading was observed in the laboratory tests (Baenziger and Porter (2011); Schultz and Hutchinson (2001)). Minimum Code requirements have been provided (Schultz (1996)) to satisfy both strength and energy criteria.

Joint reinforcement of at least 3/16 in. (4.8 mm) diameter longitudinal wire is deemed to have sufficient strain elongation and, thus, was selected as the minimum size when joint reinforcement is used as the primary shear and flexural reinforcement. The research (Baenziger and Porter (2011)) was for walls that contained a minimum of two 3/16 in. (4.8 mm) diameter longitudinal wires in a bed joint. Other research (Schultz and Hutchinson (2001)) contained two No. 9 gage (0.148 in. (3.76 mm)) diameter longitudinal wires or two No. 5 gage (0.207 in. (5.26 mm)) diameter longitudinal wires in a bed joint. The No. 5 gage longitudinal wires exhibited similar ductility to the joint reinforcement in the Baenziger/Porter research.

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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, 9, 10, 11, or 12. 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, 7.3.2.9, 7.3.2.10, or 7.3.2.11.

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, 9, 10, 11, 12, 15, or Appendix D. Nonparticipating masonry elements, except those constructed of AAC masonry, shall be reinforced in the direction of span ~~in either the horizontal or vertical direction~~ in accordance with Sections 7.4.3.1.1 and 7.4.3.1.2.

7.4.3.1.1 *Horizontal reinforcement* — In walls spanning horizontally, h~~H~~orizontal reinforcement shall be provided within 16 in. (406 mm) of the top and bottom of nonparticipating masonry walls and shall consist of one of the following:

(a) Two longitudinal wires of W1.7 (MW11) joint reinforcement spaced not more than 16 in. (406 mm) on center. The space between these wires shall be the widest that the mortar joint will accommodate.

(b) Two D2 (MD13) deformed wires spaced not more than 16 in. (406 mm) on center for walls greater than 4 in. (102 mm) in width and at least one D2 (MD13) wire spaced not more than 16 in. (406 mm) on center for walls not exceeding 4 in. (102 mm) in width. Where two deformed wires are used, the space between these wires

COMMENTARY

Additional commentary on the placement of joint reinforcement with 3/16-in. (4.8-mm) diameter longitudinal wires and constructability of masonry is provided in the commentary to Section 6.1.3.1.2.

7.4.3 *Seismic Design Category C requirements* — In addition to the requirements of Seismic Design Category B, minimum levels of reinforcement and detailing are required. The minimum provisions for improved performance of masonry construction in Seismic Design Category C must be met regardless of the method of design. Shear walls designed as part of the seismic-force-resisting system in Seismic Design Category C and higher must be designed using reinforced masonry methods because of the increased risk and expected intensity of seismic activity. Ordinary reinforced masonry shear walls, ordinary reinforced AAC masonry shear walls, intermediate reinforced masonry shear walls, special reinforced masonry shear walls, or masonry infills are required to be used.

7.4.3.1 *Design of nonparticipating elements* — Reinforcement requirements of Section 7.4.3.1 ~~are traditional for conventional concrete and clay masonry. They~~ are prescriptive in nature. The intent of this requirement is to provide structural integrity for nonparticipating masonry walls by ensuring that a minimum amount of reinforcing is present in the direction of the span should the seismic induced moment exceed the cracking strength of the masonry. AAC masonry walls differ from concrete masonry walls and clay masonry walls in that the thin-bed mortar strength and associated bond strength is typically greater than that of the AAC units. Also, the unit weight of AAC masonry is typically less than one-third of the unit weight of clay or concrete masonry, reducing seismic inertial forces. This reduced load, combined with a tensile bond strength that is higher than the strength of the AAC material itself, provides a minimum level of structural integrity. Therefore, prescriptive reinforcement is not required. All masonry walls, including non-participating AAC masonry walls, are required to be designed to resist out-of-plane forces. ~~If reinforcement is required, it must be provided in the direction of the span.~~ Permitted types of reinforcement are defined in Section 6.1.1. Commentary Section 6.1.3 provides additional information.

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shall be the widest that the mortar joint will accommodate.

(c) One No. 4 (M #13) bar or one D20 (MD129) wire spaced not more than 48 in. (1219 mm) on center.

7.4.3.1.2 *Vertical reinforcement* — In walls spanning vertically, v~~V~~vertical reinforcement shall consist of at least one No. 4 (M #13) bar or one D20 (MD129) wire spaced not more than 120 in. (3048 mm). Vertical reinforcement shall be located within 16 in. (406 mm) of the ends of masonry walls.

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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 Chapter 12. Masonry shear walls shall be designed to comply with the requirements of Section 7.3.2.3, 7.3.2.4, 7.3.2.5, 7.3.2.8, 7.3.2.10, or 7.3.2.11.

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 No. 4 (M #13) lateral ties provided in the top 5 in. (127 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* — Along each line of lateral resistance at each story, ~~at least 80~~not more than 20 percent of the lateral stiffness ~~shall~~may be provided by masonry columns~~seismic-force-resisting walls~~.

Exception: Where seismic loads are determined based on a seismic response modification factor, *R*, not greater than 1.5, columns shall be permitted to contribute more than 20 percent of the lateral stiffness along any line of resistance and may be used to provide seismic load resistance.

COMMENTARY

7.4.3.2.1 *Connections to masonry columns* — Connections must be designed to transfer forces between masonry columns and horizontal elements in accordance with the requirements of Section 4.1.4. Experience has demonstrated that connections of structural members to masonry columns are vulnerable to damage during earthquakes unless properly anchored. Requirements are adapted from previously established practice developed as a result of the 1971 San Fernando earthquake.

7.4.3.2.2 *Anchorage of floor and roof diaphragms in AAC masonry structures* — Connectors are required to be placed in grout because of the relatively low strength of connectors embedded in AAC. Different detailing options are available, but often the connectors are placed in bond beams near the top of the wall.

7.4.3.2.3 *Material requirements* — The limitation on the use of ASTM C34 structural clay tile units in the seismic-force-resisting system is based on these units' limited ability to provide inelastic strength.

7.4.3.2.4 *Lateral stiffness* — In order to accurately distribute loads in a structure subjected to lateral loading, the lateral stiffness of all structural members should be considered. Although structures may be designed to use solid or perforated shear walls for lateral-load resistance or lateral systems of other materials, columns may also be incorporated for vertical capacity. The stipulation that masonry columns~~seismic-force-resisting elements~~ provide not more than 20~~at least 80~~ percent of the lateral stiffness helps ensure that ~~additional elements, such as columns, do not significantly contribute to~~ the lateral stiffness is provided primarily by the seismic-force-resisting system. It is important in areas of high seismicity that most of the stiffness be provided by the more ductile seismic-force-resisting system and not by masonry columns that have limited ductility.

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A line of lateral resistance refers to the plan view of participating members within a vertical plane that provide resistance to seismic forces, including torsional effects.

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Potential lines of lateral resistance that do not include walls should be considered in determining whether compliance with this section has been achieved. One can evaluate whether potential lines of resistance are in fact lines of resistance for which compliance with this section is required by removing those lines of resistance from the analysis and assessing the change in forces in the remaining lines of resistance. Members offset a small amount from each other such that their displacements along the line of resistance are similar should be considered to comprise a single line of resistance. See Figure CC-7.4-1 for an illustration of lines of lateral resistance.

Based on typical design assumptions, the lateral stiffness of structural elements should be based on cracked section properties for reinforced masonry and uncracked section properties for unreinforced masonry.

The designer may opt to increase the percentage of lateral stiffness provided by masonry columns if the structure is designed to perform elastically under seismic loads. The legally adopted building code may have restrictions on the use of masonry columns to resist seismic loads. For example, ASCE/SEI 7 does not currently recognize masonry columns as a seismic-force-resisting system. In such cases it may be necessary to treat masonry columns relied upon for seismic force resistance as an alternate system for the purpose of code compliance and acceptance by the authority having jurisdiction.

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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.

COMMENTARY

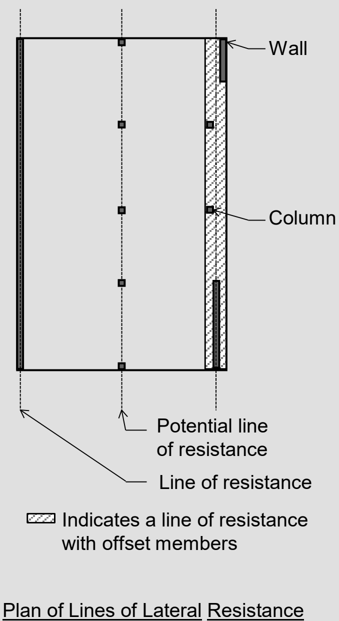

Figure CC-7.4-1 —Lines of lateral resistance

7.4.3.2.5 *Design of columns, pilasters, and beams supporting discontinuous elements* — Discontinuous stiff members such as shear walls have global overturning forces at their edges that may be supported by columns, pilasters and beams. These vertical support elements are required to have a minimum level of confinement and shear detailing at the discontinuity level. The minimum detailing requirements in this section may be in excess of those requirements that are based on calculations using full-height relative stiffnesses of the elements of the seismic-force-resisting system.

A common example is a building with internal shear walls, such as interior corridor walls, that are discontinuous at the first story above grade or in a basement level. If this structure has a rigid diaphragm at all floor and roof levels; the global (full height) relative stiffnesses of the discontinuous elements is minor in comparison to the relative stiffnesses of the continuous elements at the perimeter of the structure. All shear walls above the discontinuity, however, have a forced common interstory displacement. This forced interstory displacement induces overturning forces in the discontinuous shear walls at all levels having this forced story displacement. The accumulated overturning forces at the ends of the walls above the discontinuity in turn are likely to be supported by columns and pilasters in the discontinuous levels and the beams at the level above the discontinuity. This section

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7.4.3.2.6 *Joint reinforcement used as shear reinforcement* — The maximum spacing of horizontal joint reinforcement used as shear reinforcement in walls shall not exceed 8 in. (203 mm) in partially grouted walls. Joint reinforcement used as shear reinforcement in fully grouted walls shall consist of four 3/16 in. (4.8 mm) diameter longitudinal wires at a spacing not to exceed 8 in. (203 mm).

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, 9, 10, 11, or 12. 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 7.4.3.1.1.

(b) *Vertical reinforcement* — Vertical reinforcement shall consist of at least one No. 4 (M #13) bar or one D20 (MD 129) wire spaced not more than 48 in. (1219 mm). Vertical reinforcement shall be located within 16 in. (406 mm) of the ends of masonry walls.

7.4.4.1.1 *Minimum reinforcement for masonry columns* — Lateral ties conforming to the requirements of Section 7.4.4.2.1 shall be provided for a length equal to twice the larger column plan dimension from the top and bottom of the column at each floor.

COMMENTARY

specifies minimum detailing requirements for these columns, pilasters, and beams. The stiffness of the discontinuous element should be determined based on the relative stiffness of the discontinuous members above and below the discontinuity. If the interstory stiffness of the discontinuous wall below the discontinuity is less than 20% of the interstory stiffness above the discontinuity, the discontinuous element should be considered stiff.

7.4.3.2.6 *Joint reinforcement used as shear reinforcement* — See Commentary for Section 7.4.1.2.1.

~~**7.4.4** *Seismic Design Category D requirements* — Masonry shear walls for structures assigned to Seismic Design Category D are required to meet the requirements of special reinforced masonry shear walls or ordinary reinforced AAC masonry shear walls because of the increased risk and expected intensity of seismic activity.~~

~~The minimum amount of wall reinforcement for special reinforced masonry shear walls has been a long-standing, standard empirical requirement in areas of high seismic loading. It is expressed as a percentage of gross cross-sectional area of the wall. It is intended to improve the ductile behavior of the wall under earthquake loading and assist in crack control. Because the minimum required reinforcement may be used to satisfy design requirements, at least 1/3 of the minimum amount is reserved for the lesser stressed direction in order to ensure an appropriate distribution of loads in both directions.~~

7.4.4.1.1 *Minimum reinforcement for masonry columns* — When columns are not isolated from building displacements, yielding of reinforcing steel or crushing of masonry may occur in response to those displacements. Providing a level of confinement consistent with that required for participating columns is intended to maintain column integrity in those conditions. The length of

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Exception: Compliance with this provision is not required if either of the following requirements are met:

(a) The column is isolated from building displacements in conformance with Section 7.3.1.

(b) An analysis complying with Section 7.3.1 demonstrates that the column will remain elastic when subjected to the required inelastic displacement.

7.4.4.2 *Design of participating elements* — Masonry shear walls shall be designed to comply with the requirements of Section 7.3.2.5, 7.3.2.8, or 7.3.2.11.

7.4.4.2.1 *Minimum reinforcement for masonry columns* — Lateral ties in masonry columns shall be spaced not more than 8 in. (203 mm) on center and shall be at least 3/8 in. (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.

COMMENTARY

twice the larger column plan dimension represents the extent over which the inelastic behavior is expected to be concentrated.

This level of confinement may not be sufficient to allow the development of plastic hinges. If building displacements are to be accommodated through hinging of the non-participating columns, the rotation capacity of the columns will need to be assessed. See discussion in the commentary to Section 7.3.1 on the use of plastic hinges to accommodate building movements.

7.4.4.2 *Design of participating elements* — Masonry shear walls for structures assigned to Seismic Design Category D are required to meet the requirements of special reinforced masonry shear walls, ordinary reinforced AAC masonry shear walls, or special reinforced prestressed masonry shear walls because of the increased risk and expected intensity of seismic activity.

7.4.4.2.1 *Minimum reinforcement for masonry columns* — Adequate lateral restraint is important for column longitudinal reinforcement ~~subjected to overturning~~reisting compression forces due to earthquakes. Many column failures during earthquakes have been attributed to buckling of longitudinal reinforcement and inadequate ~~lateral tying~~ confinement of concrete or masonry in compression. For this reason, closer spacing of lateral ties than might otherwise be required is prudent. An arbitrary minimum spacing has been established through experience that provides a limited amount of ductility, consistent with an *R* value not greater than 1.5 as required by Section 7.4.3.2.4. ~~Columns not involved in the seismic force-resisting system should also be more heavily tied at the tops and bottoms for more ductile performance~~ in potential plastic hinge regions~~and better resistance to shear~~.

The larger minimum lateral tie diameter required by this provision makes it more likely that units may need to be modified to accommodate the lateral ties as is discussed in the commentary to Section 5.4.1.4.

7.4.4.2.2 *Material requirements* — Based on numerous tests by several researchers, (Brown and Melander (1999); Hamid et al (1979); Minaie et al (2009); Klingner et al (2010)) the behavior of fully grouted walls subjected to out-of-plane flexural and in-plane shear loads is dominated by grout and unaffected by mortar formulation. In tests by Minaie et al (2009) and Klingner et al (2010), fully grouted concrete masonry walls exhibited good in-plane response when subjected to seismic loads. For fully grouted participating elements in buildings assigned to Seismic Categories D or higher, no mortar material restrictions are necessary. Historical provisions requiring use of Type S or M cement-lime or mortar cement mortar are retained for partially grouted participating elements in buildings assigned to Seismic Design Categories D or higher.

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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.

7.4.5 *Seismic Design Categories E and F requirements* — Masonry elements in structures assigned to Seismic Design Category E or F shall comply with the requirements of Section 7.4.4 and with the additional requirements of Section 7.4.5.1.

7.4.5.1 *Minimum reinforcement for nonparticipating masonry elements not laid in running bond* — Masonry not laid in running bond in nonparticipating elements shall have a cross-sectional area of horizontal reinforcement of at least 0.0015 multiplied by the gross cross-sectional area of masonry, using specified dimensions. The maximum spacing of horizontal reinforcement shall be 24 in. (610 mm). These elements shall be fully grouted and shall be constructed of hollow open-end units or two wythes of solid units.

COMMENTARY

7.4.5 *Seismic Design Categories E and F requirements* — See Commentary Sections 7.3.2.2.1 and 7.4.4. The ratio of minimum horizontal reinforcement is increased to reflect the possibility of higher seismic loads. Where fully grouted open end hollow units are used, part of the need for horizontal reinforcement is satisfied by the mechanical continuity provided by the grout core.

PART 3: ENGINEERED DESIGN METHODS

CHAPTER 8
ALLOWABLE STRESS DESIGN OF MASONRY

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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~~8.1.2 through ~~8.1.5~~8.1.6, and either Section 8.2 or 8.3.

8.1.2 *Calculated stresses*

Calculated stresses shall be determined in accordance with the allowable stress design load combinations as designated in Section 4.1.2, except as noted in this Chapter.

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~~8.1.2~~8.1.3 *Design stress*

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

~~8.1.3~~8.1.4 *Anchor bolts embedded in grout*

~~8.1.3~~8.1.4.1 *Design requirements* — Anchor bolts shall be designed using either the provisions of Section ~~8.1.3~~8.1.4.2 or, for headed and bent-bar anchor bolts, by the provisions of Section ~~8.1.3~~8.1.4.3.

COMMENTARY

8.1 — General

8.1.1 *Scope*

Chapter 8 design procedures follow allowable stress design methodology, in which the calculated stresses resulting from allowable stress level loads must not exceed permissible masonry and steel stresses.

For allowable stress design, linear elastic materials following Hooke's Law are assumed, that is, deformations (strains) are linearly proportional to the loads (stresses). All materials are assumed to be homogeneous and isotropic, and sections that are plane before bending remain plane after bending. These assumptions are adequate within the low range of working stresses under consideration. The allowable stresses are fractions of the specified compressive strength, resulting in conservative factors of safety.

The stresses allowed under the action of allowable stress level loads are limited to values within the elastic range of the materials.

~~8.1.2~~8.1.3 *Design stress*

Calculated stresses designated by 'f' with subscript indicating stress type are required to be less than allowable stresses designated by 'F' with subscript indicating the same stress type.

~~8.1.3~~8.1.4 *Anchor bolts embedded in grout*

Anchor bolt designs produced using Allowable Stress Design or Strength Design should be approximately the same. See Commentary 9.1.6 for additional discussion on the background and application of the anchor bolt design provisions.

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~~8.1.3~~8.1.4.2 *Allowable loads determined by test*

~~8.1.3~~8.1.4.2.1 Anchor bolts shall be tested in accordance with ASTM C1892. Loading conditions of the test shall be representative of intended use of the anchor bolt.

~~8.1.3~~8.1.4.2.2 Anchor bolt allowable loads used for design shall not exceed 20 percent of the average failure load from the tests.

~~8.1.3~~8.1.4.3 *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~~8.1.4.3.1 through ~~8.1.3~~8.1.4.3.3.

~~8.1.3~~8.1.4.3.1 *Allowable axial tensile load of headed and bent-bar 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. The allowable axial tensile load, B_a, for bent-bar anchor bolts embedded in grout shall be the smallest of the values determined by Equations 8-1, 8-2, and 8-3.

The value of f_u shall not be taken greater than the smaller of $1.9f_y$ and 125,000 psi (862 MPa).

$$B_{ab} = 1.25 A_{pt} \sqrt{f'_m} \qquad \text{(Equation 8-1)}$$

$$B_{as} = 0.5 A_b f_u \qquad \text{(Equation 8-2)}$$

$$B_{ap} = 0.6 f'_m e_b d_b + 120\pi \left(l_b + e_b + d_b \right) d_b \qquad \text{(Equation 8-3)}$$

COMMENTARY

~~8.1.3~~8.1.4.3 *Allowable loads determined by calculation for headed and bent-bar anchor bolts* — The anchor provisions in this Code define bolt shear and tension capacities based on the bolt's specified tensile strength, f_u. The nominal strength of anchors is best represented as a function of f_u, rather than f_y, because many anchor materials do not exhibit a well-defined yield point.

The limitation of $1.9f_y$ on f_u is to ensure that, under allowable stress level loads, the anchor does not exceed f_y. The limit of f_u to a maximum of $1.9f_y$ was determined by converting strength level loads to corresponding allowable stress level loads. A load factor of 1.4 (average from $1.2D + 1.6L$ or from $1.4D$) divided by the highest ϕ-factor (0.75 for tension) results in a limit of f_u/f_y of 1.4/0.75=1.87. Although not a concern for standard structural steel anchors (maximum value of f_u/f_y is 1.7 for ASTM A307), the limitation is applicable to some stainless steels.

~~8.1.3~~8.1.4.3.1 *Allowable axial tensile load of headed and bent-bar anchor bolts* — Equation 8-1 defines the allowable axial tensile load governed by masonry breakout. Equation 8-2 defines the allowable axial tensile load governed by the tensile strength of the steel. Equation 8-3 defines the allowable axial tensile load governed by anchor pullout.

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~~8.1.3~~8.1.4.3.2 *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 Equations 8-4, 8-5, 8-6 and 8-7. The value of f_u shall not be taken greater than the smaller of $1.9f_y$ and 125,000 psi (862 MPa).

$$B_{vb} = 1.25 A_{pv} \sqrt{f'_m} \qquad \text{(Equation 8-4)}$$

$$B_{vc} = 580 \sqrt[4]{f'_m A_b} \qquad \text{(Equation 8-5)}$$

$$B_{vpry} = 2.0 B_{ab} = 2.5 A_{pt} \sqrt{f'_m} \qquad \text{(Equation 8-6)}$$

$$B_{vs} = 0.25 A_b f_u \qquad \text{(Equation 8-7)}$$

~~8.1.3~~8.1.4.3.3 *Combined axial tension and shear* — Anchor bolts subjected to axial tension in combination with shear shall satisfy Equation 8-8.

$$\left(\frac{b_a}{B_a}\right)^{5/3} + \left(\frac{b_v}{B_v}\right)^{5/3} \le 1 \qquad \text{(Equation 8-8)}$$

~~8.1.4~~8.1.5 *Shear stress in composite masonry*

~~8.1.4~~8.1.5.1 Design of composite masonry shall meet the requirements of Section 5.1.~~4~~3.2 and Section ~~8.1.4~~8.1.5.2.

~~8.1.4~~8.1.5.2 Shear stresses developed at the interfaces between wythes and collar joints or within headers shall not exceed the following:

(a) mortared collar joints, 7 psi (48.3 kPa).

(b) grouted collar joints, 13 psi (89.6 kPa).

(c) headers, $1.3\sqrt{f_h}$ psi.

COMMENTARY

~~8.1.3~~8.1.4.3.2 *Allowable shear load of headed and bent-bar anchor bolts* — Equation 8-4 defines the allowable shear load governed by masonry breakout. Equation 8-5 defines the allowable shear load governed by masonry crushing. Equation 8-6 defines the allowable shear load governed by anchor pryout. Equation 8-7 defines the allowable shear load governed by the shear strength of the steel.

~~8.1.3~~8.1.4.3.3 *Combined axial tension and shear* — The exponent of 5/3 was determined from testing by Fabrello-Streufert et al (2003) and McGinley (2006).

~~8.1.4~~8.1.5 *Shear stress in composite masonry*

Limited test data (McCarthy et al (1985); Williams and Geschwinder (1982); Colville et al (1987)) are available to document shear strength of collar joints in masonry.

Test results (McCarthy et al (1985); Williams and Geschwinder (1982)) show that shear bond strength of collar joints could vary from as low as 5 psi (34.5 kPa) to as high as 100 psi (690 kPa), depending on type and condition of the interface, consolidation of the joint, and type of loading. McCarthy et al (1985) reported an average value of 52 psi (359 kPa) with a coefficient of variation of 21.6 percent. An allowable shear stress value of 7 psi (48.3 kPa), which is four standard deviations below the average, is considered to account for the expected high variability of the interface bond. With some units, Type S mortar slushed collar joints may have better shear bond characteristics than Type N mortar. Results show that thickness of joints, unit absorption, and reinforcement have a negligible effect on shear bond strength. Grouted collar joints have higher allowable shear bond stress than the mortared collar joints (Williams and Geschwinder (1982)).

A strength analysis has been demonstrated by Porter et al (1986 and 1987) for composite masonry walls subjected to combined in-plane shear and gravity loads. In addition, these authors have shown adequate behavioral characteristics for both brick-to-brick and brick-to-block composite walls with a grouted collar joint (Wolde-Tinsae et al (1985a); Wolde-Tinsae et al (1985b); Ahmed (1983). Finite element models for analyzing the interlaminar shearing stresses in collar joints of composite walls have been investigated (Anand and Young (1982); Anand (1985); Stevens and Anand (1985); Anand and Rahman

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~~8.1.5~~8.1.6 *Bearing stress*

Bearing stresses on masonry shall not exceed 0.33 f'_m and shall be calculated over the bearing area, A_{br}, as defined in Section ~~4.3~~4.4.4.

(1986)). They found that the shear stresses were principally transferred in the upper portion of the wall near the point of load application for the in-plane loads. Thus, below a certain distance, the overall strength of the composite masonry is controlled by the global strength of the wall, providing that the wythes are acting compositely.

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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 8-9 and Equation 8-10.

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \qquad \text{(Equation 8-9)}$$

$$P \leq \left(\tfrac{1}{4}\right) P_e \qquad \text{(Equation 8-10)}$$

where:

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

$$F_a = \left(\tfrac{1}{4}\right) f'_m \left[1 - \left(\frac{h}{140r}\right)^2\right] \qquad \text{(Equation 8-11)}$$

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

$$F_a = \left(\tfrac{1}{4}\right) f'_m \left(\frac{70\,r}{h}\right)^2 \qquad \text{(Equation 8-12)}$$

COMMENTARY

8.2 — Unreinforced masonry

8.2.1 *Scope*

This section provides for the design of masonry members in which tensile stresses, not exceeding allowable limits, are resisted by the masonry. Flexural tensile stresses may result from bending moments, from eccentric vertical loads, or from lateral loads.

8.2.2 *Design criteria*

A fundamental premise is that under the effects of design loads, masonry remains uncracked. Stresses due to restraint against differential movement, temperature change, moisture expansion, and shrinkage combine with the design load stresses. Stresses due to restraint should be controlled by joints or other construction techniques to ensure that the combined stresses do not exceed the allowable.

8.2.3 *Design assumptions*

Reinforcement may be placed in masonry walls to control the effects of movements from temperature changes or shrinkage, or as prescriptive seismic reinforcement. This reinforcement is not considered in calculating strength when using unreinforced masonry design.

8.2.4 *Axial compression and flexure*

8.2.4.1 *Axial and flexural compression* — Equation 8-9 is a unity interaction equation that is a simple proportioning of the available allowable stresses to the applied loads, and the equation is used to design masonry for combined axial and flexural compressive stresses. The unity equation can be expanded when biaxial bending is present by adding a third term for the bending stress quotient about the second axis of bending.

In this unity interaction equation, secondary bending effects resulting from the axial load are ignored. A more accurate equation would include the use of a moment magnifier applied to the flexure term, f_b/F_b. Although avoidance of a moment magnifier term can produce unconservative results in some cases, the Committee decided not to include this term in Equation 8-9 for the following reasons:

- At larger h/r values, where moment magnification is more critical, the allowable axial load on the member is limited by Equation 8-10.
- For the practical range of h/r values, errors induced by ignoring the moment magnifier is relatively small, less than 15 percent.

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(c) $F_b = \left(\frac{1}{3}\right) f'_m$ (Equation 8-13)

(d) $P_e = \dfrac{\pi^2 E_m I_n}{h^2}\left(1 - 0.577\dfrac{e}{r}\right)^3$ (Equation 8-14)

COMMENTARY

- The overall safety factor of 4 included in the allowable stress equations is sufficiently large to allow this simplification in the design procedure.

The purpose of Equation 8-10 is to safeguard against a premature stability failure caused by eccentrically applied axial load. The equation is not intended to be used to check adequacy for combined axial compression and flexure. Therefore, in Equation 8-14, the value of the eccentricity "*e*" that is to be used to calculate P_e is the actual eccentricity of the applied compressive load. The value of "*e*" is not to be calculated as M_{max} divided by P where M_{max} is a moment caused by other than eccentric load.

Equation 8-10 is an essential check because the allowable compressive stress for members with an *h/r* ratio in excess of 99 has been developed assuming only a nominal eccentricity of the compressive load. Thus, when the eccentricity of the compressive load exceeds the minimum eccentricity of 0.1*t*, Equation 8-12 will overestimate the allowable compressive stress and Equation 8-10 may control.

The allowable stress values for F_a presented in Equations 8-11 and 8-12 are based on an analysis of the results of axial load tests performed on clay masonry members and concrete masonry members. A fit of an empirical curve to this test data, Figure CC-8.2-1, indicates that members having an *h/r* ratio not exceeding 99 fail under loads below the Euler buckling load at a stress level equal to:

$$f'_m\left[1 - \left(\frac{h}{140r}\right)^2\right] \quad \text{(same with SI units)}$$

Thus, for members having an *h/r* ratio not exceeding 99, this Code allows axial load stresses not exceeding $^1/_4$ of the aforementioned failure stress.

Applying the Euler theory of buckling to members having resistance in compression but not in tension, (Colville (1978); Colville (1979); Yokel (1971)) show that for a solid section, the critical compressive load for these members can be expressed by the formula

$$P_e = \left(\pi^2 E_m I_n / h^2\right)\left(1 - 2e/t\right)^3 \quad \text{(same with SI units)}$$

in which

I_n = uncracked moment of inertia

e = eccentricity of axial compressive load with respect to the member longitudinal centroidal axis.

In the derivation of this buckling load equation, tension cracking is assumed to occur prior to failure.

For *h/r* values in excess of 99, the limited test data is approximated by the buckling load.

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COMMENTARY

For a solid rectangular section, $r = \sqrt{t^2 / 12}$. Making this substitution into the buckling load equation gives

$$P_e = \frac{\pi^2 E_m I_n}{h^2}\left(1\text{-}0.577\frac{e}{r}\right)^3 \qquad \text{(Equation 8-14)}$$

Transforming the buckling equation using a minimum eccentricity of $0.1t$ (from Section 8.3.4.3) and an elastic modulus equal to $1000 f'_m$, the axial compressive stress at buckling failure amounts approximately to $[70(r / h)]^2 f'_m$. At the time of the development of this equation, the Committee had not developed a relationship between E_m and f'_m so the traditional relationship of $E_m = 1000 f'_m$ was used (Colville (1992)). The same equation can be developed using $E_m = 667 f'_m$ and an eccentricity of $0.05t$. Thus, for members having an h/r ratio in excess of 99, this Code allows an axial load compressive stress not exceeding $^1/_4$ of this failure stress (Equation 8-12).

Tests of masonry have shown (Hatzinikolas et al (1978); Fattal and Cattaneo (1976); Yokel and Dikkers (1971); Yokel and Dikkers (1973)) that the maximum compressive stress at failure under flexural load is higher than the maximum compressive stress at failure under axial load. The higher stress under flexural load is attributed to the restraining effect of less highly strained compressive fibers on the fibers of maximum compressive strain. This effect is less pronounced in hollow masonry than solid masonry; however, the test data indicate that, calculated by the straight-line theory, the compressive stress at failure in hollow masonry subjected to flexure exceeds by $^1/_3$ that of the masonry under axial load. Thus, to maintain a factor of safety of 4 in design, the Committee considered it conservative to establish the allowable compressive stress in flexure as:

$$F_b = \tfrac{4}{3} \times \left(\tfrac{1}{4}\right) f'_m = \left(\tfrac{1}{3}\right) f'_m$$

COMMENTARY

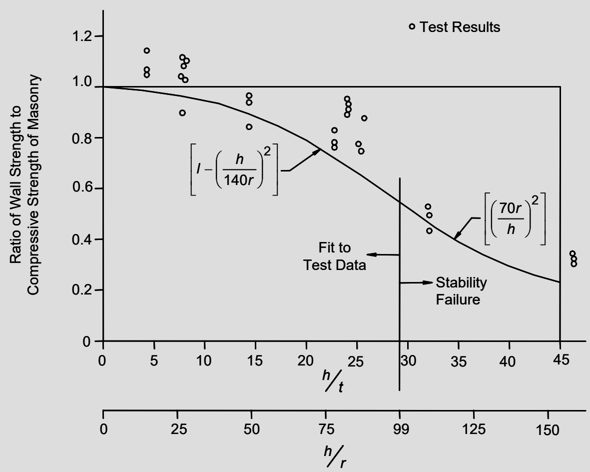

Figure CC-8.2-1 — Slenderness effects on axial compressive strength

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8.2.4.2 *Flexural tension* — Allowable tensile stresses for masonry subjected to out-of-plane or in-plane bending, other than structural clay tile, shall be in accordance with the values in Table 8.2.4.2. 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.

COMMENTARY

8.2.4.2 *Flexural tension* — Prior to the 2011 edition of this Code, allowable stresses were permitted to be increased by one-third when considering load combinations including wind or seismic loads. Unreinforced masonry walls designed under codes that permitted the one-third stress increase have had acceptable performance. However, rather than arbitrarily increasing the allowable flexural tensile stresses by one-third, the Committee assessed the allowable flexural tensile stresses using a reliability-based approach to see if an increase in allowable stresses is justified. Kim and Bennett (2002) performed a reliability analysis in which the flexural tensile stress was assumed to follow a lognormal distribution. They used a mean flexural tensile strength of the allowable flexural tensile stress in the 2008 Code multiplied by 5.1 based on the examination of 327 full-scale tests reported in the literature. Coefficients of variations for different data sets (e.g. specific mortar type and direction of loading) ranged from 0.10 to 0.51, with a weighted average of 0.42. The coefficient of variation of 0.50 used by Kim and Bennett (2002) is greater than used in previous studies. For example, Ellingwood et al (1980) used a coefficient of variation of 0.24 and Stewart and Lawrence (2000) used a coefficient of variation of 0.30. Kim and Bennett felt, though, that a coefficient of variation of 0.50 is more representative of field conditions. The lognormal distribution was determined by comparing the Anderson-Darling statistic for normal, lognormal, and Weibull probability distributions. For unreinforced masonry walls subjected to wind loading and designed using the one-third stress increase, the reliability

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index was determined to be 2.66. This is slightly greater than the value of 2.5 that is typical for the design of other materials (Ellingwood et al, 1980). The reliability analysis by Kim and Bennett (2002) assumed the axial load was zero, which is the worst case. With increasing axial load (which has a lower coefficient of variation than 0.50), the reliability index would increase. Based on this reliability analysis, the Committee felt justified in increasing the allowable flexural tensile stresses by a factor of 4/3 to compensate for the elimination of the previously permitted one-third stress increase.

The allowable tensile strength values are a function of the type of mortar being used. Mortar cement is a product that has bond strength requirements that have been established to provide comparable flexural bond strength to that achieved using portland cement-lime mortar (Melander and Ghosh (1996); Hedstrom et al (1991); Borchelt and Tann (1996)).

For masonry cement and air entrained portland-cement lime mortar, there are no conclusive research data and, hence, flexural tensile stresses are based on existing requirements in other codes.

The allowable tensile stresses are for tension stresses due to flexure under either out-of-plane or in-plane loading. While it is recognized that in-plane and out-of-plane strain gradients are different, for these low stress levels the effect due to any difference is small. Flexural tensile stress can be offset by axial compressive stress, but the net tensile stress due to combined bending and axial compression cannot exceed the allowable flexural tensile stress.

Variables affecting tensile bond strength of brick masonry normal to bed joints include mortar properties, unit initial rate of absorption, surface condition, workmanship, and curing condition. For tension parallel to bed joints, the strength and geometry of the units also affect tensile strength.

Historically, masonry not laid in running bond has been assumed to have no flexural bond strength across mortared head joints; thus the grout area alone is used to resist bending. Examples of continuous grout parallel to the bed joints are shown in Figure CC-8.2-2.

Table 8.2.4.2: Allowable flexural tensile stresses for clay[1] and concrete masonry, psi (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	53 (366)	40 (276)	32 (221)	20 (138)
Hollow units [2]				
Ungrouted	33 (228)	25 (172)	20 (138)	12 (83)
Fully grouted	65 (448)	63 (434)	61 (420)	58 (400)
Parallel to bed joints in running bond				
Solid units	106 (731)	80 (552)	64 (441)	40 (276)
Hollow units				
Ungrouted and partially grouted	66 (455)	50 (345)	40 (276)	25 (172)
Fully grouted	106 (731)	80 (552)	64 (441)	40 (276)
Parallel to bed joints in masonry not laid in running bond				
Continuous grout section parallel to bed joints	133 (917)	133 (917)	133 (917)	133 (917)
Other	0 (0)	0 (0)	0 (0)	0 (0)

1 The values in this table shall not be applicable to structural clay tile unit masonry (ASTM C34, ASTM C56, ASTM C126, ASTM C212).

2 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.

COMMENTARY

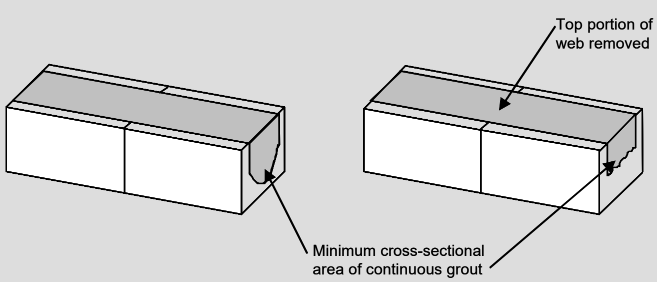

Figure CC-8.2-2 — Continuous grout sections parallel to the bed joints

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COMMENTARY

Test data using a bond wrench (Brown and Palm (1982); Hamid (1985)) revealed tensile bond strength normal to bed joints ranging from 30 psi (207 kPa) to 190 psi (1,310 kPa). This wide range is attributed to the multitude of parameters affecting tensile bond strength.

Test results (Hamid (1985); Ribar (1982)) show that masonry cement mortars and mortars with high air content generally have lower bond strength than portland cement-lime mortars.

Tests conducted by Hamid (1981) show the significant effect of the aspect ratio (height to least dimension) of the brick unit on the flexural tensile strength. The increase in the aspect ratio of the unit results in an increase in strength parallel to bed joints and a decrease in strength normal to bed joints.

While the principles of Chapter 8 can be used to design structural clay tile masonry, the values in Table 8.2.4.2 are not applicable to masonry constructed with structural clay tile units. Plummer (1962, 1977) reports some flexural tension values for masonry composed of these units and industry sources may also provide values.

Research work (Drysdale and Hamid (1984)) on flexural strength of concrete masonry has shown that grouting has a significant effect in increasing tensile strength over ungrouted masonry. A three-fold increase in tensile strength normal to bed joints was achieved using fine grout as compared to ungrouted masonry. The results also show that, within a practical range of strength, the actual strength of grout is not of major importance. For tension parallel to bed joints, a 133 percent increase in flexural strength was achieved by grouting the cells. Grout cores change the failure mode from stepped-wise cracking along the bed and head joints for hollow walls to a straight line path along the head joints and unit for grouted walls.

Research (Brown and Melander (1999)) has shown that flexural strengths normal to the bed joint of unreinforced, fully grouted clay masonry and concrete masonry are largely independent of mortar type or cementitious materials.

For partial grouting, the footnote to Table 8.2.4.2 permits interpolation between the fully grouted value and the hollow unit value based on the percentage of grouting. A concrete masonry wall with Type S portland cement-lime mortar grouted 50 percent and stressed normal to the bed joints would have an allowable stress midway between 65 psi (448 kPa) and 33 psi (228 kPa), hence an allowable stress of 49 psi (338 kPa).

The presence of flashing and other conditions at the base of the wall can significantly reduce the flexural bond. The values in Table 8.2.4.2 apply only to the flexural tensile stresses developed between masonry units, mortar, and grout.

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8.2.5 *Axial tension*

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

8.2.6 *Shear*

Members shall be designed in accordance with Sections 8.2.6.1 through 8.2.6.3.

8.2.6.1 Shear stresses due to forces acting in the direction considered shall be calculated in accordance with Section ~~4.3~~4.4.1 and determined by Equation 8-15.

$$f_v = \frac{VQ}{Ib} \qquad \text{(Equation 8-15)}$$

COMMENTARY

8.2.5 *Axial tension*

Net axial tension in unreinforced masonry walls due to axially applied load is not permitted. If axial tension develops in walls due to uplift of connected roofs or floors, the walls must be reinforced to resist the tension. Compressive stress from dead load can be used to offset axial tension.

8.2.6 *Shear*

Three modes of shear failure in unreinforced masonry are possible:

(a) Diagonal tension cracks form through the mortar and masonry units.

(b) Sliding occurs along a straight crack at horizontal bed joints.

(c) Cracks form, that stair-step from head joint to bed joint.

For simplicity, the Committee recommends that the allowable shear stress values given in Section 8.2.6.2 be used for limiting both in-plane and out-of-plane shear stresses.

8.2.6.1 A theoretical parabolic stress distribution is used to define shear stress through the depth. Some other codes use average shear stress so direct comparison of allowable values is not valid. Effective area requirements are given in Section ~~4.3~~4.4.1. For rectangular sections, Equation 8-15 produces a maximum shear stress at mid-depth, that is equal to $^3/_2 \times V/A$. Equation 8-15 is also used to calculate shear stresses for composite action and shear stresses, resulting from out-of-plane loading, in the connections between face shells of hollow units.

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8.2.6.2 Allowable shear stresses for masonry shall not exceed the smallest of:

(a) $1.5\sqrt{f'_m}$

(b) 120 psi (0.827 MPa)

(c) the applicable values given in Table 8.2.6.2

8.2.6.3 The minimum normalized web area of concrete masonry units, determined in accordance with ASTM C140, shall not be less than 25 in.^{2}/ft^2 (173,600 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).

COMMENTARY

8.2.6.2 Shear stress allowable values are applicable to shear walls without reinforcement. The values given are based on research (Woodward and Rankin (1984); Pook (1986); Nuss et al (1978); Hamid et al (1979)).

Masonry that is not laid in running bond, does not have horizontal reinforcement, and is not fully grouted with open-ended units (both ends) will not transfer significant shear stress across head joints. In these cases, however, the minimum prescriptive horizontal reinforcing that is required in Section ~~4.6~~4.7 helps to maintain the structural integrity of the head joints and an allowable shear stress of 15 psi (0.103 MPa) is permitted.

8.2.6.3 Out-of-plane flexure causes horizontal and vertical shear stresses. Vertical shear stresses are resisted by the connection between the web and face shell of the unit. A normalized web area of 25 in.^{2}/ft^2 (173,600 mm^2/m^2) provides sufficient web area so that shear stresses between the web and face shell of a unit, resulting from out-of-plane loading, will not be critical.

Table 8.2.6.2: Allowable Shear Stresses for Clay and Concrete Masonry, F_v, psi (MPa)

Masonry Bond Pattern	Construction	
	Fully Grouted	**Other than Fully Grouted**
Masonry Laid in Running Bond	$60 + 0.45\ P/A_n$ $(0.414 + 0.45\ P/A_n)$	$37 + 0.45\ P/A_n$ $(0.255 + 0.45\ P/A_n)$
Masonry Not Laid in Running Bond		
Open-Ended Units	$37 + 0.45\ P/A_n$ $(0.255 + 0.45\ P/A_n)$	15 (0.103)
Other than Open-Ended Units	15 (0.103)	15 (0.103)

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8.3 — Reinforced masonry

8.3.1 *Scope*

This section provides requirements for the design of structures in which reinforcement is used in conjunction with the masonry to resist forcest. 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~~5.4.1.4.

(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: 20,000 psi (137.9 MPa)

(b) Grade 60 reinforcement: 32,000 psi (220.7 MPa)

8.3.3.2 Tensile stress in joint reinforcement, deformed wire, and welded deformed wire reinforcement shall not exceed 30,000 psi (206.9 MPa).

8.3.3.3 When lateral reinforcement is provided in compliance with the requirements of Section ~~5.3~~5.4.1.4, 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.

COMMENTARY

8.3 — Reinforced masonry

8.3.1 *Scope*

The requirements in this section pertain to the design of masonry in which flexural tension is assumed to be resisted by reinforcement alone, and the flexural tensile strength of masonry is neglected.

8.3.2 *Design assumptions*

The design assumptions listed have traditionally been used for allowable stress design of reinforced masonry members.

Although tension may develop in the masonry of a reinforced member, it is not considered effective in resisting axial and flexural design loads.

8.3.3 *Steel reinforcement — Allowable stresses —* The allowable steel stresses have a sufficiently large factor of safety that second-order effects do not need to be considered in allowable stress design.

8.3.4 *Axial compression and flexure*

See Commentary for Section 8.2.4.1.

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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 8-16 or Equation 8-17:

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

$$P_a = \left(0.30 f'_m A_n + 0.65 A_{st} F_s\right)\left[1-\left(\frac{h}{140r}\right)^2\right]$$

(Equation 8-16)

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

$$P_a = \left(0.30 f'_m A_n + 0.65 A_{st} F_s\right)\left(\frac{70r}{h}\right)^2$$

(Equation 8-17)

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 0.45 f'_m.

COMMENTARY

8.3.4.2 *Allowable forces and stresses* — This Code limits the compressive stress in masonry members based on the type of load acting on the member. The compressive force at the section resulting from axial loads or from the axial component of combined loads is calculated separately, and is limited to the values permitted in Section 8.3.4.2.1. Equation 8-16 or 8-17 controls the capacity of columns with large axial loads. The coefficient of 0.30 provides a safety level approximately equivalent to that for compression-controlled sections in strength design. The coefficient of 0.65 was determined from tests of reinforced masonry columns and is taken from previous masonry codes (ACI 531 (1983); BIA (1969)). A second compressive stress calculation must be performed considering the combined effects of the axial load component and flexure at the section and should be limited to the values permitted in Section 8.3.4.2.2. (See Commentary for Section 8.2.4.)

8.3.4.2.2 Figure CC-8.3-1 shows the allowable moment (independent of member size and material strength) versus the ratio of steel reinforcement (Grade 60) multiplied by the steel yield strength and divided by the specified compressive strength of masonry (modified steel reinforcement ratio) for both clay and concrete masonry members subjected to pure flexure. When the masonry compressive stress controls the design, there is little increase in moment capacity with increasing steel reinforcement. This creates a limit on the amount of reinforcement that is practical to use in allowable stress design of masonry. Even when the masonry allowable compressive stress controls the design, the failure of the member will still be ductile. For clay masonry, the masonry stress begins to control the design at $0.39\rho_{bal}$ and for concrete masonry, the masonry stress begins to control the design at $0.38\rho_{bal}$, where ρ_{bal} is the reinforcement ratio at which the masonry would crush simultaneously with yielding of the reinforcement. The reinforcement ratio as a fraction of the balanced reinforcement ratio, ρ_{bal}, is also shown in Figure CC-8.3-1.

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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.

COMMENTARY

The interaction equation used in Section 8.2.4 is not applicable for reinforced masonry and is therefore not included in Section 8.3.

8.3.4.3 *Columns* — The minimum eccentricity of axial load (Figure CC-8.3-2) results from construction imperfections not otherwise anticipated by analysis.

In the event that actual eccentricity exceeds the minimum eccentricity required by this Code, the actual eccentricity should be used. This Code requires that stresses be checked independently about each principal axis of the member (Figure CC-8.3-2).

Additional column design and detailing requirements are given in Section ~~5.3~~5.4.

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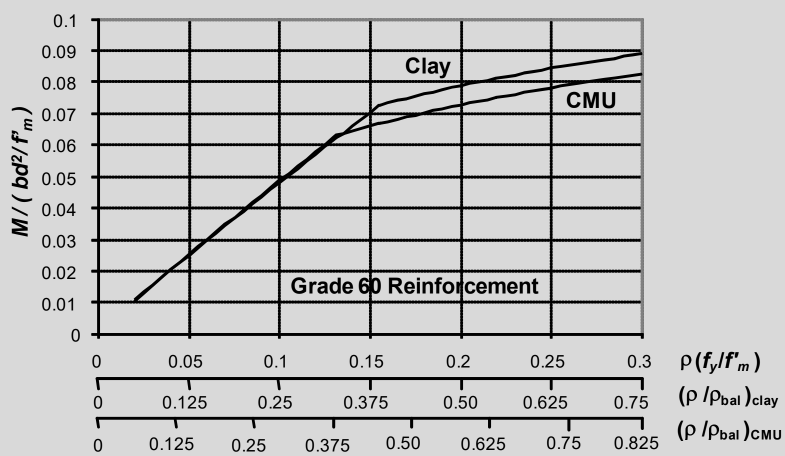

Figure CC-8.3-1 Allowable moment vs. modified steel reinforcement ratio

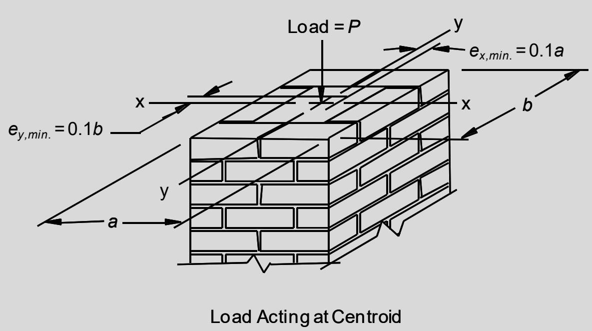

Figure CC-8.3-2 — Minimum design eccentricity in columns

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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 $0.05f'_mA_n$, shall have a maximum ratio of flexural tensile reinforcement, ρ_{max}, not greater than that calculated as follows:

$$\rho_{\max} = \frac{nf'_m}{2f_y\left(n+\dfrac{f_y}{f'_m}\right)} \qquad \text{(Equation 8-18)}$$

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:

$$f_v = \frac{V}{A_{nv}} \qquad \text{(Equation 8-19)}$$

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 8-20 and shall not be taken greater than the limits given by Section 8.3.5.1.2 (a) through (c).

$$F_v = \left(F_{vm} + F_{vs}\right)\gamma_g \qquad \text{(Equation 8-20)}$$

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

$$F_v \leq \left(3\sqrt{f'_m}\right)\gamma_g \qquad \text{(Equation 8-21)}$$

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

$$F_v \leq \left(2\sqrt{f'_m}\right)\gamma_g \qquad \text{(Equation 8-22)}$$

COMMENTARY

8.3.4.4 *Walls* — The balanced reinforcement ratio for a masonry wall with a single layer of reinforcement designed by allowable stress design can be derived by applying principles of engineering mechanics to a cracked, transformed section. The resulting equation is:

$$\rho_b = \frac{nF_b}{2F_s\left(n+\dfrac{F_s}{F_b}\right)}$$

where ρ_b is the balanced reinforcement ratio resulting in a condition in which the reinforcement and the masonry simultaneously reach their specified allowable stresses. However, the ratio of allowable steel tensile stress to the specified yield strength of the reinforcement, and the ratio of allowable masonry compressive stress to the specified compressive strength of the masonry are not consistent (F_s can range from 40 percent to 53 percent of f_y while F_b is taken equal to $0.45f'_m$). Therefore, allowable stresses in the equation above are replaced with the corresponding specified strengths, as shown in Equation 8-18.

The equation is directly applicable for reinforcement concentrated at the end of the shear wall. For distributed reinforcement, the reinforcement ratio is obtained as the total area of tension reinforcement divided by bd.

8.3.5 *Shear*

Prior to the 2011 edition of the Code, the shear resistance provided by the masonry was not added to the shear resistance provided by the shear reinforcement (in allowable stress design). A recent study (Davis et al (2010)) examined eight different methods for predicting the in-plane shear capacity of masonry walls. The design provisions of Chapter 9 (strength design) of this Code were found to be the best predictor of shear strength. Therefore, the provisions of Chapter 9, which allow for the shear resistance provided by the masonry to be added to the shear resistance provided by the shear reinforcement, were appropriately modified and adopted for Chapter 8. See the flow chart for design of masonry members resisting shear shown in Figure CC-8.3-3.

8.3.5.1.2 Allowable shear stress Equations 8-20 through 8-22 are based on strength design provisions, but reduced by a factor of safety of 2 to obtain allowable stress values. The provisions of this Section were developed through the study of and calibrated to cantilevered shear walls. The ratio $M/(Vd_v)$, can be difficult to interpret or apply consistently for other conditions such as for a uniformly loaded, simply supported beam. Concurrent values of M and Vd_v must be considered at appropriate locations along shear members, such as beams, to determine the critical $M/(Vd_v)$ ratio. To simplify the analytical process, designers are permitted to use $M/(Vd_v) = 1$. Commentary Section 9.3.4.1.2 provides additional information. Partially grouted shear walls can have lower strengths than predicted by the shear capacity equations using just the reduction of net area (Minaie et al (2010); Nolph and ElGawady (2011); Schultz (1996a); Schultz

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γ_g = 0.70 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 8-23.

$$F_{vm} = \frac{1}{2}\left[\left(4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] + 0.20\frac{P}{A_n} \geq 0$$

(Equation 8-23)

$M/(Vd_v)$ shall be taken as a positive number and need not be taken greater than 1.0. P shall be considered positive for net compressive axial loads and negative for net tensile axial loads.

8.3.5.1.4 The allowable shear stress resisted by the steel reinforcement, F_{vs}, shall be calculated using Equation 8-24:

$$F_{vs} = 0.5\left(\frac{A_v F_s d_v}{A_{nv} s}\right)$$ (Equation 8-24)

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 48 in. (1219 mm).

8.3.5.2.2 For shear walls, 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 8 ft (2.44 m).

COMMENTARY

(1996b); Schultz and Hutchinson (2001)). The grouted shear wall factor, γ_g, was introduced in the 2013 edition of TMS 402 to compensate for this reduced capacity. A statistical analysis (Dillon and Fonseca (2017)) conducted on a large dataset of results from fully and partially grouted shear wall tests has indicated that a grouted wall factor value of γ_g = 0.70 for partially grouted walls produces factors of safety similar to those for fully grouted masonry walls. Commentary Section 9.3.3.1.2 provides additional information.

8.3.5.1.3 Equation 8-23 is based on strength design provisions with the masonry shear strength reduced by a factor of safety of 2 and allowable stress level loads used instead of strength level loads.

8.3.5.1.4 Commentary Section 9.3.4.1.2.2 provides additional information.

8.3.5.2.1 The assumed shear crack is at 45 degrees to the longitudinal reinforcement. Thus, a maximum spacing of $d/2$ is specified to assure that each crack is crossed by at least one bar or wire. The 48 in. (1219 mm) maximum spacing is an arbitrary choice that has been in codes for many years.

COMMENTARY

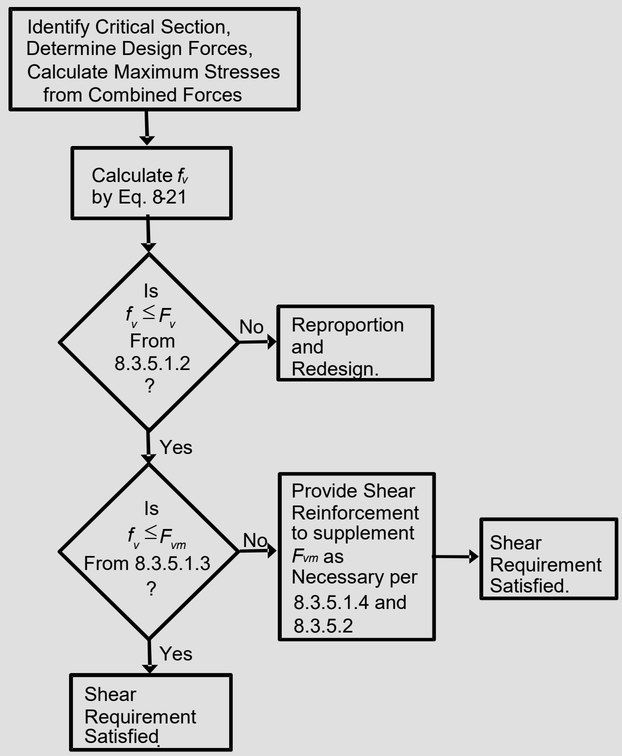

Figure CC-8.3-3 — Flow chart for shear design

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8.3.6 *Shear-Friction*

Provisions of this section shall apply to shear transfer across horizontal interfaces in walls subjected to in-plane loads. The shear stress in a horizontal interface shall be calculated with Equation 8-19 and shall not exceed the allowable shear-friction stress, F_f, where F_f shall be determined as follows.

Where $M/(Vd_v) \le 0.5$,

$$F_f = \frac{\mu\left(A_{sp}F_s + P\right)}{A_{nv}} \ge 0 \qquad \text{(Equation 8-25)}$$

The coefficient of friction, μ, shall be 1.0 for masonry on concrete with an unfinished surface, or masonry on concrete with a finished surface that has been intentionally roughened; μ shall be 0.70 for all other conditions.

Where $M/(Vd_v) \ge 1.0$,

$$F_f = \frac{0.65\left(0.75A_{sp}F_s + P\right)}{A_{nv}} \ge 0 \qquad \text{(Equation 8-26)}$$

Where $M/(V\,d_v)$ is between 0.5 and 1.0, the value of F_f shall be determined by linear interpolation between the values given by Equations 8-25 and 8-26.

The reinforcement considered in Equation 8-25 and 8-26 shall be adequately anchored above and below the horizontal shear plane to develop the yield strength of the reinforcement. The value of P is negative in Equations 8-25 and 8-26 when it is a tension force.

COMMENTARY

8.3.6 *Shear-Friction*

When subjected to in-plane lateral loads, walls that have a low axial compressive load and a low shear-span ratio are vulnerable to shear sliding, which normally occurs at the base. Shear sliding can cause severe damage in masonry due to the simultaneous actions of the shear stress, compressive stress, and dowel force; it can weaken lap splices adjacent to the shear plane; it can fracture the reinforcement crossing the shear plane. Provisions of this section are to prevent shear sliding. Equation 8-25 is essentially identical to Equation 9-31 except that the nominal yield strength of the reinforcement is replaced by the allowable stress. Equation 8-26 accounts for the fact that for flexure-dominated walls, not all the reinforcement crossing the horizontal shear plane will contribute to the clamping force and the coefficient of friction for masonry under high compressive stress will be reduced. The reduced friction coefficient is assumed to be 0.65 regardless of the surface condition. Equations 8-25 and 8-26 are based on the same considerations as Equations 9-31 and 9-32, which are for the calculation of the shear-friction resistance in strength design as explained in the Commentary Section 9.3.5.5.

The coefficient of friction is a function of the roughness of the surface. Historically, the Uniform Building Code (1997) required concrete abutting structural masonry to be roughened to a full amplitude of 1/16 inch (1.6 mm). Unfinished concrete and finished concrete that is intentionally roughened will meet this requirement.

In flanged shear walls, only the webs are included in the net shear area, A_{nv}. Therefore only web reinforcement is considered effective in shear friction resistance and is included in A_{sp} reinforcement.

CHAPTER 9
STRENGTH DESIGN OF MASONRY

TMS 402 CODE

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 the legally adopted building code~~as designated in Section 4.1.2, except as noted in this Chapter. Members subject to compressive axial load shall be designed for the strength level moment accompanying the strength level axial load. The strength level moment, M_u, shall include the moment induced by relative lateral displacement.

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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*

COMMENTARY

9.1 — General

9.1.1 *Scope*

Chapter 9 design procedures follow strength design methodology, in which internal forces resulting from application of strength level loads must not exceed design strength (nominal member strength reduced by a strength-reduction factor ϕ).

Materials are assumed to be homogenous, isotropic, and exhibit nonlinear behavior. Under loads that exceed service levels, nonlinear material behavior, cracking, and reinforcement slip invalidate the assumption regarding the linearity of the stress-strain relation for masonry, grout, and reinforcing steel. If nonlinear behavior is modeled, however, nominal strength can be accurately predicted.

Much of the substantiating data for the strength design criteria in this Chapter was provided by research conducted by the Technical Coordinating Committee for Masonry Research (TCCMaR). This research program resulted in 63 research reports from 1985-1992. These reports are available from The Masonry Society, Longmont, CO. A summary of the TCCMaR program is found in Noland and Kingsley (1995).

9.1.3 *Design strength*

Nominal member strengths are typically calculated using minimum specified material strengths.

9.1.4 *Strength-reduction factors*

The strength-reduction factor accounts for the uncertainties in construction, material properties, calculated versus actual member strengths, as well as anticipated mode of failure. Strength-reduction (ϕ) factors are assigned values based on limiting the probability of failure to an acceptably small value, with some adjustment based on judgment and experience.

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9.1.4.1 *Anchor bolts* — Strength reduction factor, ϕ, shall be as follows based on the governing nominal anchor bolt strength:

(a) Tensile strength of anchor steel 0.75

(b) Shear strength of anchor steel 0.65

(c) Anchor pullout 0.65

(d) Masonry breakout, masonry crushing, or anchor pryout .... 0.50.

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 for reinforced masonry subjected to flexure, axial load, or combinations thereof shall be in accordance with Table 9.1.4.

COMMENTARY

9.1.4.1 *Anchor bolts* — Because of the similarity between the behavior of anchor bolts embedded in grout and in concrete, and because available research data for anchor bolts in grout indicate similarity, the strength-reduction values associated with various controlling anchor bolt failures are derived from expressions based on research into the performance of anchor bolts embedded in concrete. In the 2022 edition of this Code, anchor bolt strength was changed to be based on f_u instead of f_y. Although the ϕ-factors for use with fu appear low, they result in a level of safety consistent with the use of higher factors applied to f_{yu}.

9.1.4.2 *Bearing* — The value of the strength-reduction factor used in bearing assumes that some degradation has occurred within the masonry material.

9.1.4.3 *Combinations of flexure and axial load in unreinforced masonry* — The same strength-reduction factor is used for the axial load and the flexural tension or compression induced by bending moment in unreinforced masonry. The lower strength-reduction factor associated with unreinforced masonry (in comparison to reinforced masonry) reflects an increase in the coefficient of variation of the measured strengths of unreinforced masonry when compared to similarly configured reinforced masonry.

9.1.4.4 *Combinations of flexure and axial load in reinforced masonry* — The nominal strength of a member that is subjected to flexure, axial load, or a combination thereof is determined for a strain condition where the strain in the extreme compression fiber is the maximum usable compressive strain of the masonry ε_{mu}, and the strain in the extreme tension reinforcement is the net tensile strain ε_t. The value of ε_t is determined from a linear strain distribution at nominal strength.

Members subjected to only axial compression are considered to be compression-controlled and members subjected to only axial tension are considered to be tension-controlled.

For sections subjected to combined flexure and axial load, design strengths are determined by multiplying both P_n and M_n by the value of ϕ determined from Table 9.1.4. For sections within the transition region, the value of ϕ is determined by linear interpolation, as shown in Figure CC-9.1-1.

For lightly reinforced masonry walls under out-of-plane loads, it is possible to have design axial strengths in the tension-controlled and transition region that are greater than the design strength at the beginning of the compression-controlled region, as shown by the dashed line in Figure CC-9.1-2. To prevent the odd-shaped design strength interaction diagram, the design axial strength in the tension-controlled and transition region is limited to the design axial strength at the beginning of the compression-controlled region, or at balanced conditions, as shown by the solid line in Figure CC-9.1-2. It can be shown that Section 9.1.4.4.2 will never control for fully grouted sections with a single layer of tension reinforcement, and for fully grouted shear walls subjected to in-plane loads with uniformly distributed reinforcement. Section 9.1.4.4.2 can

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COMMENTARY

control for lightly reinforced walls under out-of-plane loads, particularly for thicker masonry units.

Table CC-9.1-1 gives limiting values of the factored axial load, P_u, such that sections are tension-controlled. As long as P_u is less than these values, the strength-reduction factor will be 0.90. If the value of P_u in Table CC-9.1-1 is negative, this means that a tension force is needed for the section to be tension-controlled. The section would be compression-controlled or in the transition region for all compressive axial loads. The value of A_s for fully grouted shear walls is the total area of vertical reinforcement along the wall. The limits for fully grouted shear walls can also be used for partially grouted shear walls if the width b is adjusted to account for the amount of grouting. For partially grouted walls subjected to out-of-plane loads, b_w is the width of the compression section minus the sum of the length of ungrouted cells, and t_{fs} is the specified face-shell thickness for hollow masonry units. The equations are based on a yield strain of 0.002.

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9.1.4.4.1 The value of ε_{ty} shall be f_y/E_s. For Grade 60 reinforcement it shall be permitted to take ε_{ty} equal to 0.002.

9.1.4.4.2 In the tension-controlled and transition regions, the value of ϕ for axial load shall be limited so that $\phi P_n \leq 0.65 P_{bal}$, where P_{bal} is determined using a strain gradient corresponding to a strain in the extreme tensile reinforcement equal to ε_{ty} and a maximum strain in the masonry as given by Section 9.3.2(c).

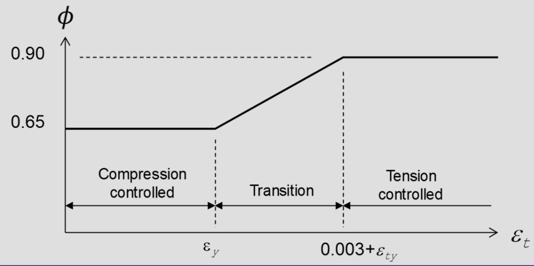
9.1.4.5 *Shear and Shear-Friction* — The value of ϕ shall be taken as 0.80 for shear and shear-friction design.

9.1.4.5 *Shear and Shear Friction* — The strength-reduction factor for calculating the design shear strength recognizes the greater uncertainty in calculating nominal shear strength than in calculating nominal flexural strength.

Table 9.1.4: Strength reduction factor ϕ for moment, axial load, or combined moment and axial load

Net tensile strain, ε_t	Classification	ϕ
$\varepsilon_t \leq \varepsilon_{ty}$	Compression-controlled	0.65
$\varepsilon_{ty} < \varepsilon_t < 0.003+\varepsilon_{ty}$	Transition	$0.65+0.25\dfrac{\varepsilon_t-\varepsilon_{ty}}{0.003}$
$\varepsilon_t \geq 0.003+\varepsilon_{ty}$	Tension-controlled	0.90

COMMENTARY



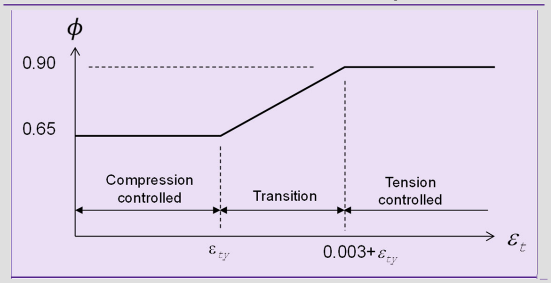


Figure CC-9.1-1 Variation of ϕ with net tensile strain in extreme tension reinforcement, ε_t

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Table CC-9.1-1: Limiting Value of P_u for tension-controlled sections

Masonry Element	Concrete Masonry	Clay Masonry
Fully grouted section with single layer of tension reinforcement	$P_u \leq 0.19 f'_m bd - 0.9 A_s f_y$	$P_u \leq 0.24 f'_m bd - 0.9 A_s f_y$
Fully grouted shear wall subjected to in-plane loads with uniformly distributed reinforcement	$P_u \leq 0.19 f'_m bd_v - 0.48 A_s f_y$	$P_u \leq 0.24 f'_m bd_v - 0.42 A_s f_y$
Partially grouted wall with a single layer of tension reinforcement subjected to out-of-plane loads	$P_u \leq 0.19 f'_m bd - 0.9 A_s f_y$ for $t_{fs} \geq 0.27d$ $P_u \leq 0.72 f'_m \left(bt_{fs} + \left(0.27d - t_{fs}\right)b_w\right) - 0.9 A_s f_y$ for $t_{fs} < 0.27d$ but not greater than $P_u \leq 0.52 f'_m \left(bt_{fs} + \left(0.44d - t_{fs}\right)b_w\right) - 0.65 A_s f_y$	$P_u \leq 0.24 f'_m bd - 0.9 A_s f_y$ for $t_{fs} \geq 0.33d$ $P_u \leq 0.72 f'_m \left(bt_{fs} + \left(0.33d - t_{fs}\right)b_w\right) - 0.9 A_s f_y$ for $t_{fs} < 0.33d$ but not greater than $P_u \leq 0.52 f'_m \left(bt_{fs} + \left(0.50d - t_{fs}\right)b_w\right) - 0.65 A_s f_y$

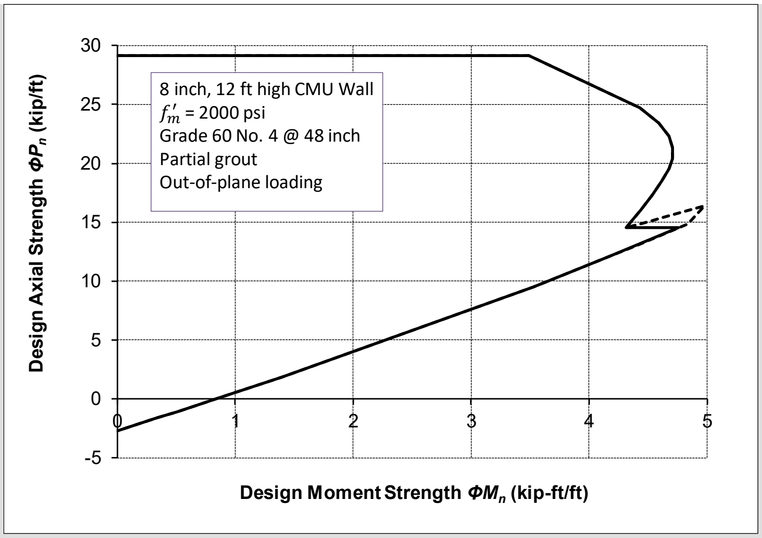

Figure CC-9.1-2 Design strength interaction diagram for a lightly reinforced masonry wall

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9.1.5 *Deformation requirements*

9.1.5.1 *Deflection of unreinforced masonry* — Deflection calculations for unreinforced 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 uncracked section properties, unless a cracked-section analysis is performed.

9.1.6 *Anchor bolts embedded in grout*

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 C1892. 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.

COMMENTARY

9.1.5 *Deformation requirements*

9.1.5.1 *Deflection of unreinforced masonry* — The deflection calculations of unreinforced masonry are based on elastic performance of the masonry assemblage as outlined in the design criteria of Section 9.2.2.

9.1.5.2 *Deflection of reinforced masonry* — Values of I_{eff} are typically about one-half of the uncracked section of common configurations of members. Calculating a more accurate value using the cracked transformed section may be desirable for some circumstances (Abboud et al (1993); Hamid et al (1990)).

9.1.6 *Anchor bolts embedded in grout*

Design of anchor bolts embedded in grout may be based on physical testing or, for headed and bent-bar anchor bolts, by calculation. Due to the wide variation in configurations of post-installed anchors, designers are referred to product literature published by manufacturers for these anchors.

9.1.6.2 *Nominal strengths determined by test* — Many types of anchor bolts, such as expansion anchors, toggle bolts, sleeve anchors, etc., are not addressed by Section 9.1.6.3 and, therefore, such anchors must be designed using test data. Testing may also be used to establish higher strengths than those calculated by Section 9.1.6.3. ASTM C1892 requires three tests for steel failures and five tests for masonry failures. The variability of anchor bolt strength in masonry and the possibility that anchor bolts may be used in a non-redundant manner warrants the minimum of five tests stipulated for masonry failures. Assuming a normal probability distribution and a coefficient of variation of 20 percent for the test data, a fifth-percentile value for nominal strength is 67 percent, which is rounded to 65 percent of the average strength value. Failure modes obtained from testing should be reported and the associated ϕ factors should be used when establishing design strengths.

9.1.6.3 *Nominal strength determined by calculation for headed and bent-bar anchor bolts* — Design equations provided in this Code stem from research (Brown and Whitlock (1983); Hatzinikolos et al (1980); Rad et al (1998); Tubbs et al (1999); Allen et al (2000); Brown et al (2001); Weigel et al (2002)) conducted on headed anchor bolts and bent-bar anchor bolts (J- or L-bolts) embedded in grout.

The anchor provisions in this Code define bolt shear and tension capacities based on the bolt's specified tensile strength. Commentary Section ~~8.1.3~~8.1.4.3 provides further information.

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9.1.6.3.1 *Axial tensile strength of headed and bent-bar anchor bolts* — The nominal axial tensile strength, B_{an}, 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 the tensile strength of the steel). The design axial tensile strength, ϕB_{an}, shall be the smaller of the design strengths obtained by multiplying the applicable ϕ value by the nominal strength from Equations 9-1 and 9-2.

The nominal axial tensile strength, B_{an}, for bent-bar anchor bolts embedded in grout shall be determined by Equation 9-1 (nominal axial tensile strength governed by masonry breakout), Equation 9-2 (nominal axial tensile strength governed by the tensile strength of the steel), or Equation 9-3 (nominal axial tensile strength governed by anchor bolt pullout). The design axial tensile strength, ϕB_{an}, shall be the smallest of the design strengths obtained by multiplying the applicable ϕ value by the nominal strength from Equations 9-1, 9-2, and 9-3. The value of f_u shall not be taken greater than the smaller of $1.9f_y$ and 125,000 psi (862 MPa).

$$B_{anb} = 4A_{pt}\sqrt{f'_m} \qquad \text{(Equation 9-1)}$$

$$B_{ans} = A_b f_u \qquad \text{(Equation 9-2)}$$

$$B_{anp} = 1.5 f'_m e_b d_b + 300\pi\left(\ell_b + e_b + d_b\right)d_b \qquad \text{(Equation 9-3)}$$

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 9-4 (nominal shear strength governed by masonry breakout), Equation 9-5 (nominal shear strength governed by masonry crushing), Equation 9-6 (nominal shear strength governed by anchor bolt pryout) or Equation 9-7 (nominal shear strength governed by the shear strength of the steel). The design shear strength ϕB_{vn}, shall be the smallest of the design strengths obtained by multiplying the applicable ϕ value by the nominal strength from Equations 9-4, 9-5, 9-6, and 9-7.

The value of f_u shall not be taken greater than the smaller of $1.9f_y$ and 125,000 psi (862 MPa).

$$B_{vnb} = 4A_{pv}\sqrt{f'_m} \qquad \text{(Equation 9-4)}$$

$$B_{vnc} = 1750\sqrt[4]{f'_m A_b} \qquad \text{(Equation 9-5)}$$

$$B_{vnpry} = 2.0B_{anb} = 8A_{pt}\sqrt{f'_m} \qquad \text{(Equation 9-6)}$$

$$B_{vns} = 0.6A_b f_u \qquad \text{(Equation 9-7)}$$

COMMENTARY

9.1.6.3.1 *Axial tensile strength of headed and bent-bar anchor bolts* — Tensile strength of a headed anchor bolt is governed by breakout of an approximately conical volume of masonry starting at the anchor head and having a fracture surface oriented at approximately 45 degrees to the masonry surface, Equation 9-1, or by the tensile strength of the anchor steel, Equation 9-2.

Tensile strength of a bent-bar anchor bolt (J- or L-bolt) is governed by tensile cone breakout of the masonry, Equation 9-1, by the tensile strength of the anchor steel, Equation 9-2, or by straightening and pullout of the anchor bolt from the masonry, Equation 9-3. The second term in Equation 9-3 is the portion of the anchor bolt capacity due to bond between bolt and grout. Accordingly, TMS 602 Article 3.2 A requires that precautions be taken to ensure that the shanks of bent-bar anchor bolts are clean and free of debris that would otherwise interfere with the bond between anchor bolt and grout.

9.1.6.3.2 *Shear strength of headed and bent-bar anchor bolts* — Shear strength of a headed or bent-bar anchor bolt is governed by the shear strength of the anchor steel, Equation 9-7, by masonry crushing, Equation 9-5, or by masonry shear breakout, Equation 9-4. Equation 9-7 assumes the portion of the bolt passing through the shear plane is threaded. Pryout (see Figure CC-6.3-7) is also a possible failure mode. The pryout equation (Equation 9-6) is adapted from concrete research (Fuchs et al (1995)).

Under static shear loading, bent-bar anchor bolts do not exhibit straightening and pullout. Under reversed cyclic shear, however, available research (Malik et al (1982)) suggests that straightening and pullout may occur.

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9.1.6.3.3 *Combined axial tension and shear* — Anchor bolts subjected to axial tension in combination with shear shall satisfy Equation 9-8.

$$\left(\frac{b_{au}}{\phi B_{an}}\right)^{5/3}+\left(\frac{b_{vu}}{\phi B_{vn}}\right)^{5/3}\leq 1 \qquad \text{(Equation 9-8)}$$

9.1.7 *Shear strength in composite masonry*

9.1.7.1 Design of composite masonry shall meet the requirements of Sections 5.1.~~4~~3.2 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, 14 psi (96.5 kPa).

(b) grouted collar joints, 26 psi (179.3 kPa).

(c) headers, $2.6\sqrt{f_h}$ psi.

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~~4.4.

9.1.9 *Material properties*

~~**9.1.9.1** *Compressive strength*~~

~~**9.1.9.1.1** *Masonry compressive strength* — The value of f'_m used to determine nominal strength values in this chapter shall not exceed 4,000 psi (27.58 MPa) for concrete masonry and shall not exceed 6,000 psi (41.37 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 5,000 psi (34.47 MPa). For clay masonry, the specified compressive strength of grout, f'_g, shall not exceed 6,000 psi (41.37 MPa).~~

COMMENTARY

9.1.6.3.3 *Combined axial tension and shear* — Commentary Section ~~8.1.3~~8.1.4.3.3 provides additional information.

9.1.7 *Shear strength in composite masonry*

The nominal shear strength is based on shear stresses that are twice the allowable shear stresses in allowable stress design. Commentary Section ~~8.1.4~~8.1.5 provides additional information.

9.1.8 *Nominal bearing strength*

Commentary Section 4.~~3~~4.4 provides further information.

9.1.9 *Material properties*

Commentary Section 4.2 provides additional information.

~~**9.1.9.1** *Compressive strength*~~

~~**9.1.9.1.1** *Masonry compressive strength* — Design criteria are based on TCCMaR research (Noland and Kingsley (1995)) conducted on structural masonry components having compressive strength in the range of 1,500 to 4,000 psi (10.34 to 27.58 MPa) for concrete masonry and 1,500 to 6,000 psi (10.34 to 41.37 MPa) for clay masonry. Thus, the upper limits given represent the upper values that were tested in the research. The code does not explicitly stipulate a minimum specified compressive strength for application with its design provisions. Compliance with the material requirements of TMS 602 implicitly establish a minimum masonry compressive strength. Care should be used when applying these provisions to materials and assemblies that do not conform to the requirements of TMS 602.~~

~~**9.1.9.1.2** *Grout compressive strength* — Because most empirically derived design equations calculate nominal strength as a function of the specified compressive strength of the masonry, the specified compressive strength of the grout is required to be at least equal to the specified compressive strength for concrete masonry. This requirement is an attempt to ensure that where the grout compressive strength controls the design (such as anchors embedded in grout), the nominal strength will not be affected. The limitation on the maximum grout~~

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~~9.1.9.2~~9.1.9.1 *Masonry modulus of rupture* — The modulus of rupture, f_r, for masonry subjected to out-of-plane or in-plane bending, other than structural clay tile, shall be in accordance with the values in Table ~~9.1.9.2~~9.1.9.1. 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.

COMMENTARY

~~compressive strength is due to the lack of available research using higher material strengths.~~

~~9.1.9.2~~9.1.9.1 *Masonry modulus of rupture* — The modulus of rupture values provided in Table ~~9.1.9.2~~9.1.9.1 are directly proportional to the allowable stress values for flexural tension. In-plane and out-of-plane strain gradients are recognized as being different, but at these low stress levels this effect should be small.

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While the principles of Chapter 9 may be used to design structural clay tile masonry, the values in Table ~~9.1.9.2~~9.1.9.1 are not applicable to masonry constructed with structural clay tile units. Plummer (1962, 1977) reports some modulus of rupture values for masonry composed of these units and industry sources may also provide values.

Historically, masonry not laid in running bond has been assumed to have no flexural bond strength across mortared head joints; thus, the grout area alone is used to resist bending. Examples of a continuous grout section parallel to the bed joints are shown in Figure CC-8.2-2.

The presence of flashing and other conditions at the base of the wall can significantly reduce the flexural bond. The values in Table ~~9.1.9.2~~9.1.9.1 apply only to the flexural tensile stresses developed between masonry units, mortar, and grout.

Table ~~9.1.9.2~~9.1.9.1: Modulus of rupture[1], f_r, psi (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	133 (919)	100 (690)	80 (552)	51 (349)
Hollow units[2]				
Ungrouted	84 (579)	64(441)	51 (349)	31 (211)
Fully grouted	163 (1124)	158 (1089)	153 (1055)	145 (1000)
Parallel to bed joints in running bond				
Solid units	267 (1839)	200 (1379)	160 (1103)	100 (689)
Hollow units				
Ungrouted and partially grouted	167 (1149)	127 (873)	100 (689)	64 (441)
Fully grouted	267 (1839)	200 (1379)	160 (1103)	100 (689)
Parallel to bed joints in masonry not laid in running bond				
Continuous grout section parallel to bed joints	335 (2310)	335 (2310)	335 (2310)	335 (2310)
Other	0 (0)	0 (0)	0 (0)	0 (0)

[1] The values in this table shall not be applicable to structural clay tile unit masonry (ASTM C34, ASTM C56, ASTM C126, ASTM C212)

[2] 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.

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~~9.1.9.3~~9.1.9.2 *Reinforcement strengths*

~~9.1.9.3~~9.1.9.2.1 *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 60,000 psi (413.7 MPa). The actual yield strength shall not exceed 1.3 multiplied by the specified yield strength.

~~9.1.9.3~~9.1.9.2.2 *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 60,000 psi (413.7 MPa) for reinforcing bars and which shall not exceed 85,000 psi (586 MPa) for reinforcing wire.

COMMENTARY

~~9.1.9.3~~9.1.9.2 *Reinforcement strengths*

~~9.1.9.3~~9.1.9.2.1 *Reinforcement for in-plane flexural tension and flexural tension perpendicular to bed joints* — TCCMaR Research (Noland and Kingsley (1995)) conducted on reinforced masonry components used Grade 60 reinforcement. To be consistent with laboratory documented investigations, design is based on a nominal steel yield strength of 60,000 psi (413.7 MPa). The limitation on the flexural steel yield strength of 130 percent of the nominal yield strength is to minimize the over-strength unintentionally incorporated into a design.

~~9.1.9.3~~9.1.9.2.2 *Reinforcement for in-plane shear and flexural tension parallel to bed joints* — The limitation on steel yield strength of 130 percent of the nominal yield strength, in Section ~~9.1.9.3~~9.1.9.2.1, does not apply to shear reinforcement because the risk of brittle shear failures is reduced with higher yield strength.

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9.2 — Unreinforced 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 masonry cross-sections for combined flexure and axial loads shall be determined so that:

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

(b) the tensile stress does not exceed the modulus of rupture determined from Section ~~9.1.9.2~~9.1.9.1.

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:

$$P_n = 0.80\left\{0.80 A_n f'_m\left[1-\left(\frac{h}{140r}\right)^2\right]\right\}$$

(Equation 9-9)

COMMENTARY

9.2 — Unreinforced masonry

9.2.1 *Scope*
Commentary Section 8.2.1 provides further information.

9.2.2 *Design criteria*
The design of unreinforced masonry requires that the structure performs elastically under design loads. The system response factors used in the design of unreinforced masonry assume an elastic response. Commentary Section 8.2.2 provides further information.

9.2.3 *Design assumptions*
Commentary Section 8.2.3 provides further information.

9.2.4 *Nominal flexural and axial strength*
9.2.4.1 *Nominal strength* — This section gives requirements for constructing an interaction diagram for unreinforced masonry members subjected to combined flexure and axial loads. The requirements are illustrated in Figure CC-9.2-1. Also shown in Figure CC-9.2-1 are the requirements of Section 9.2.4.2, which give a maximum axial force.

9.2.4.2 *Nominal axial strength* — Commentary Section 9.3.3.1.1 gives additional information.

COMMENTARY

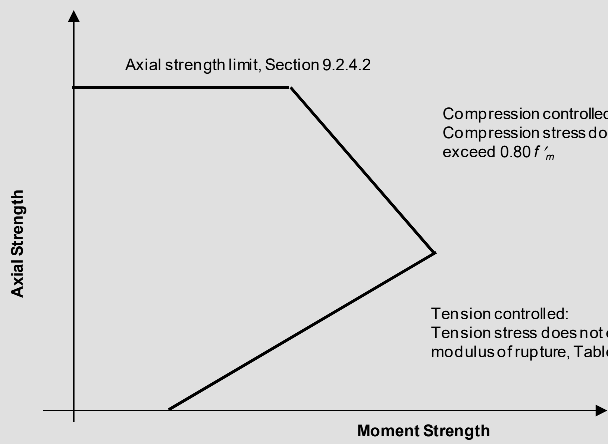

Figure CC-9.2-1 Interaction diagram for unreinforced masonry members

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

$$P_n = 0.80\left[0.80 A_n f'_m \left(\frac{70r}{h}\right)^2\right] \quad \text{(Equation 9-10)}$$

9.2.4.3 *P-Delta effects*

9.2.4.3.1 Members shall be designed for the strength level 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 Equations 9-11 and 9-12.

$$M_u = \psi M_{u,0} \quad \text{(Equation 9-11)}$$

$$\psi = \frac{1}{1 - \dfrac{P_u}{A_n f'_m \left(\dfrac{70r}{h}\right)^2}} \quad \text{(Equation 9-12)}$$

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

COMMENTARY

9.2.4.3 *P-delta effects* — *P*-delta effects are either determined by a second-order analysis, which includes *P*-delta effects, or a first-order analysis, which excludes *P*-delta effects and the use of moment magnifier. The moment magnifier is determined as:

$$\psi = \frac{C_m}{1 - \dfrac{P_u}{\varphi_k P_{euler}}}$$

where ϕ_k is a stiffness reduction factor or a resistance factor to account for variability in stiffness, C_m is a factor relating the actual moment diagram to an equivalent uniform moment diagram, and P_{euler} is Euler's buckling load. For reinforced concrete design, a value of ϕ_k = 0.75 is used (Mirza et al (1987)).

Euler's buckling load is obtained as $P_{euler} = \pi^2 E_m A_n r^2 / h^2$. Using $E_m = 700 f'_m$, which is the lower value of clay and concrete masonry, Euler's buckling load becomes:

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9.2.4.3.4 A value of $\psi = 1$ shall be permitted for members in which $45 < h/r \le 60$, 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.

COMMENTARY

$$P_{euler} = \frac{\pi^2 E_m A_n r^2}{h^2}$$

$$= \frac{\pi^2 700 f'_m A_n r^2}{h^2} = A_n f'_m \left(\frac{83.1r}{h}\right)^2$$

Current design provisions calculate the axial strength of walls with h/r>99 as $A_n f'_m (70r/h)^2$. Section 8.2.4.1 of the Commentary gives the background of this equation. It is based on using E_m=1000f'_m, neglecting the tensile strength of the masonry, and considering an accidental eccentricity of 0.10t. In spite of the fact that this equation was developed using a higher modulus than in the current Code, the equation gives a strength of $(70/83.1)^2 = 0.71$ of Euler's buckling load for clay masonry. The value of 0.71 is approximately the value of ϕ_k that has been used as a stiffness reduction factor. For ease of use and because of designer's familiarity, a value of (70 r / h) is used for Euler's buckling load instead of an explicit stiffness reduction factor. For most walls, C_m = 1. The moment magnifier can thus be determined as:

$$\psi = \frac{1}{1 - \dfrac{P_u}{A_n f'_m \left(\dfrac{70r}{h}\right)^2}}$$

Figure CC-9.2-2 shows the ratio of the second-order stress, $\dfrac{P_u}{A_n} + \dfrac{\delta M_u}{S_n}$, divided by the first-order stress, $\dfrac{P_u}{A_n} + \dfrac{M_u}{S_n}$, when the second-order stress is at the strength design limit $\varphi(0.8 f'_m)$. Typically slenderness effects are ignored if they contribute less than 5 percent (MacGregor et al (1970)). From Figure CC-9.2-2, slenderness effects contribute less than 5 percent for values of $h/r \le 45$. An intermediate wall is one with a slenderness h/r greater than 45 but not greater than 60. Slenderness effects contribute about 10 percent to the design at h/r = 60. Intermediate walls can be designed using either the moment magnifier approach or a simplified method in which the nominal stresses are reduced by 10 percent. The Code requires walls with h/r > 60 to be designed using the moment magnifier approach.

9.2.5 *Axial tension*

Commentary Section 8.2.5 provides further information.

COMMENTARY

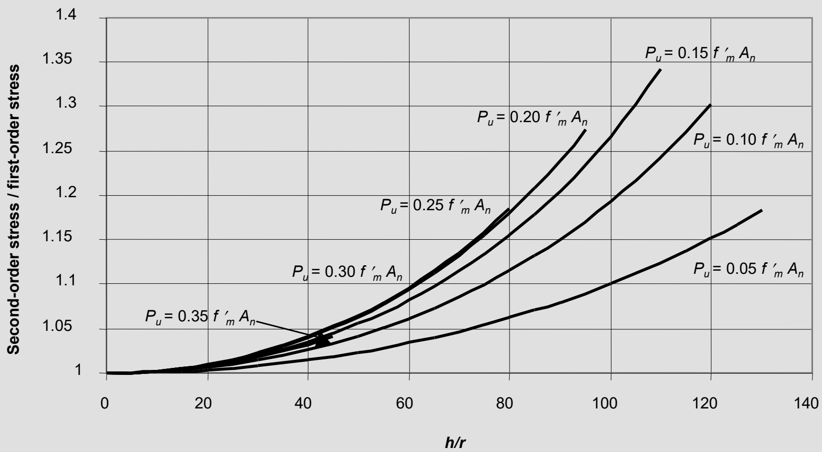

Figure CC-9.2-2 Ratio of second-order stress to first-order stress

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9.2.6 *Nominal shear strength*

9.2.6.1 Nominal shear strength of unreinforced masonry, V_n, shall not exceed the smallest of :

(a) $3.8A_{nv}\sqrt{f'_m}$

(b) $300A_{nv}$

(c) the applicable values given in Table 9.2.6.1

9.2.6.2 The minimum normalized web area of concrete masonry units, determined in accordance with ASTM C140, shall not be less than 25 in.^{2}/ft^2 (173,600 mm^2/m^2) or the nominal shear strength based on the unit web shall not exceed $3.8\sqrt{f'_m}I_n b_{web}/Q$

COMMENTARY

9.2.6 *Nominal shear strength*

9.2.6.1 For a rectangular cross-section, the shear stress is assumed to follow a parabolic distribution. The Code is based on an average shear stress, which is two-thirds of the maximum shear stress for a parabolic shear stress distribution. Commentary Section 8.2.6 provides further information.

9.2.6.2 Out-of-plane flexure causes horizontal and vertical shear stresses. Vertical shear stresses are resisted by the connection between the web and face shell of the unit. A normalized web area of 25 in.^{2}/ft^2 (173,600 mm^2/m^2) provides sufficient web area so that shear stresses between the web and face shell of a unit, resulting from out-of-plane loading, will not be critical. For simplicity, the same nominal out-of-plane shear strength as for in-plane shear is conservatively used, although peak shear stresses instead of average shear stresses are being checked.

Table 9.2.6.1: Nominal Shear Strength, V_n, of Unreinforced Clay and Concrete Masonry, lb (N)

Masonry Bond Pattern	Construction	
	Fully Grouted	**Other than Fully Grouted**
Masonry Laid in Running Bond	$90\,A_{nv} + 0.45\,P_u$ $(0.620\,A_{nv} + 0.45\,P_u)$	$56\,A_{nv} + 0.45\,P_u$ $(0.386\,A_{nv} + 0.45\,P_u)$
Masonry Not Laid in Running Bond		
Open-Ended Units	$56\,A_{nv} + 0.45\,P_u$ $(0.386\,A_{nv} + 0.45\,P_u)$	$23\,A_{nv}$ $(0.159\,A_{nv})$
Other than Open-Ended Units	$23\,A_{nv}$ $(0.159\,A_{nv})$	$23\,A_{nv}$ $(0.159\,A_{nv})$

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9.3 — Reinforced masonry

9.3.1 *Scope*

This section provides requirements for the design of structures in which reinforcement is used in conjunction with the masonry to resist forces. Design of reinforced masonry by the strength design method shall comply with the requirements of Part 1, Part 2, Section 9.1, 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 9.3.5.6.1 (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~~4.1.4.

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(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 0.80 f'_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 = 0.80 c 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.

COMMENTARY

9.3 — Reinforced masonry

9.3.1 *Scope*

The high tensile strength of reinforcement complements the high compressive strength of masonry. Increased strength and greater ductility result from the use of reinforcement in masonry structures as compared with unreinforced masonry.

9.3.2 *Design assumptions*

The design assumptions listed have traditionally been used for strength design of reinforced masonry members.

The values for the maximum usable strain are based on research on masonry materials (Assis and Hamid (1990); Brown (1987)). Concern has been raised as to the implied precision of the values. However, the Committee agrees that the reported values for the maximum usable strain reasonably represent those observed during testing.

Although tension may develop in the masonry of a reinforced member, it is not considered in calculating axial and flexural strengths.

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9.3.3 *Design of beams 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 and columns. The effects of cracking on member stiffness shall be considered.

9.3.3.1 *Nominal strength*

9.3.3.1.1 *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 9-13 or Equation 9-14, as appropriate.

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

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

(Equation 9-13)

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

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

(Equation 9-14)

9.3.3.1.2 *Nominal shear strength* — Nominal shear strength, V_n, shall be calculated using Equation 9-15, and shall not be taken greater than the limits given by Section 9.3.3.1.2 (a) through (c).

$$V_n = \left(V_{nm} + V_{ns}\right)\gamma_g \qquad \text{(Equation 9-15)}$$

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

$$V_n \le \left(6 A_{nv} \sqrt{f'_m}\right)\gamma_g \qquad \text{(Equation 9-16)}$$

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

$$V_n \le \left(4 A_{nv} \sqrt{f'_m}\right)\gamma_g \qquad \text{(Equation 9-17)}$$

γ_g = 0.70 for partially grouted shear walls and 1.0 otherwise.

(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.

COMMENTARY

9.3.3 *Design of beams and columns*

9.3.3.1 *Nominal strength*

9.3.3.1.1 *Nominal axial and flexural strength* — The nominal flexural strength of a member may be calculated using the assumption of an equivalent rectangular stress block as outlined in Section 9.3.2. Commentary Section 8.2.4 gives further information regarding slenderness effects on axial load strength as taken into account with the use of Equation 9-13 and Equation 9-14. Equation 9-13 and Equation 9-14 apply to simply supported end conditions, with or without transverse loading, which result in a symmetric deflection (curvature) about the midheight of the member. Where other support conditions or loading scenarios are known to exist, Equation 9-13 and Equation 9-14 should be modified accordingly to account for the effective height of the member or shape of the bending moment diagram over the clear span of the member. The weak-axis radius of gyration should be used in calculating slenderness-dependent reduction factors. The first coefficient, 0.80, in Equation 9-13 and Equation 9-14 accounts for unavoidable minimum eccentricity in the axial load.

9.3.3.1.2 *Nominal shear strength* — The shear strength equations in Section 9.3.3.1.2 are derived from research (Shing et al (1990a); Shing et al (1990b)). The provisions of this Section were developed through the study of and calibrated to cantilevered shear walls. The ratio $M_u/(V_u d_v)$ can be difficult to interpret or apply consistently for other conditions such as for a uniformly loaded, simply supported beam. Concurrent values of M_u and $V_u d_v$ must be considered at appropriate locations along shear members, such as beams, to determine the critical $M_u/(V_u d_v)$ ratio. To simplify the analytical process, designers are permitted to use $M_u/(V_u d_v) = 1$.

The limitations on maximum nominal shear strength are included to preclude critical (brittle) shear-related failures.

Partially grouted walls can produce lower strengths than predicted by the shear strength equations using just the reduction of net area (Minaie et al (2010); Nolph and ElGawady (2011); Schultz (1996b); Schultz (1996c); Schultz and Hutchinson (2001)). The grouted shear wall factor, γ_g, was introduced in the 2013 edition of TMS 402 to compensate for this reduced strength. A statistical analysis of the equations was conducted on the ratios of experimental to calculated strengths from tests of 167 fully grouted and 215 partially grouted masonry walls failing in in-plane shear (Dillon and Fonseca (2017)). The test data encompassed both

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9.3.3.1.2.1 *Nominal masonry shear strength* — Shear strength provided by the masonry, V_{nm}, shall be calculated using Equation 9-18:

$$V_{nm} = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_{nv}\sqrt{f'_m} + 0.25P_u \geq 0$$

(Equation 9-18)

$M_u/(V_u\, d_v)$ shall be taken as a positive number and need not be taken greater than 1.0. P_u shall be considered positive for net compressive axial loads and negative for net tensile axial loads.

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

$$V_{ns} = 0.5\left(\frac{A_v}{s}\right) f_y\, d_v \quad \text{(Equation 9-19)}$$

COMMENTARY

concrete masonry walls and clay masonry walls. The shear equations were shown to exhibit greater variability for partially grouted walls than for fully grouted walls. The analysis indicated that using $\gamma_g = 0.75$ for partially grouted walls resulted in the probability of failure for partially grouted walls being 43% greater than fully grouted walls. The difference in probability reduced to 10% when $\gamma_g = 0.70$ was used for partially grouted walls.

9.3.3.1.2.1 *Nominal masonry shear strength* — Equation 9-18 is empirically derived from research (Shing et al (1990a); Shing et al (1990b)).

9.3.3.1.2.2 *Nominal shear strength provided by reinforcement* — The nominal shear strength provided by horizontal reinforcement, as given in Equation 9-19, is an empirical equation based on a best-fit analysis of fully-grouted masonry shear wall tests performed in the 1980s (Anderson and Priestley (1992)). While predominantly an empirical relationship, the form of Equation 9-19 can be approximately represented by the assumptions and free-body diagrams shown in Figure CC-9.3-1, where only horizontal forces are considered~~shown for clarity~~. For walls with $h \geq d_v$, the equilibrium of the free body shown in Figure CC-9.3-1(a) results in V_{ns} = (number of bars or wires) $A_v f_y$. Assuming a 45-degree shear crack, the number of bars or wires is d_v/s resulting in $0.5V_{ns} = \left(\frac{A_v}{s}\right) f_y d_v$. For $h < d_v$, and assuming the shear is distributed uniformly across the top and across the bottom of the wall, the equilibrium of the free body shown in Figure CC-9.3-1(b) results in $\frac{V_{ns}}{d_v}(h+x) = (\text{number of bars or wires}) A_v f_y + \frac{V_{ns}}{d_v} x$, or $V_{ns} = (\text{number of bars}) A_v f_y \frac{d_v}{h}$. Assuming a 45-degree shear crack, the number of bars is h/s, resulting in $V_{ns} = \left(\frac{A_v}{s}\right) f_y d_v$. The empirical coefficient of 0.5 in Equation 9-19 can be considered to account~~accounts~~ for the fact that not all the horizontal reinforcement may reach the yield strength when the nominal shear capacity of a wall is reached, partly due to the fact that reinforcement near the top or the bottom of a shear crack may not have adequate development lengths to develop the yield strength (Shing et al (1990a); Shing et al (1990b)). Other coefficients were

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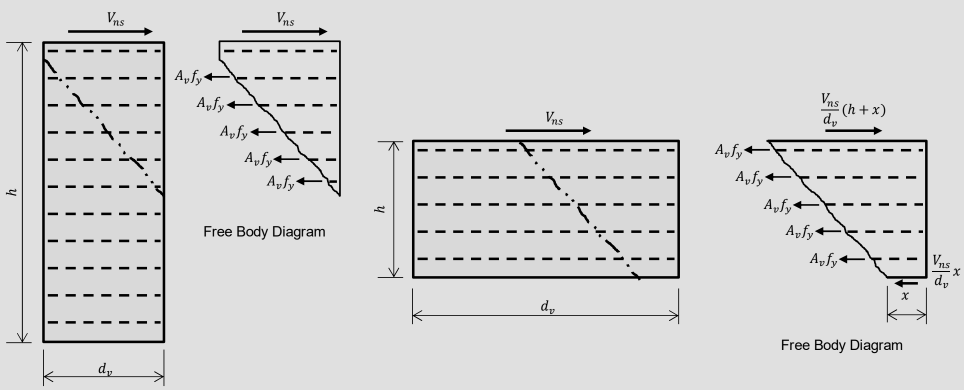
COMMENTARY

evaluated (0.6, 0.8, and 1.0), but the best fit to the experimental data was obtained using the 0.5 factor (Davis et al (2010)).

COMMENTARY



(a) $h \geq d_v$ (b) $h < d_v$

Figure CC-9.3-1 —Nominal shear strength provided by shear reinforcement

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9.3.3.2 *Beams* — Design of beams shall meet the requirements of Section 5.~~2~~3 and the additional requirements of Sections 9.3.3.2.1 through 9.3.3.2.4.

9.3.3.2.1 The strength level axial compressive load on a beam shall not exceed 0.05 $A_n f'_m$.

9.3.3.2.2 *Longitudinal reinforcement*

9.3.3.2.2.1 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~~9.1.9.1.

9.3.3.2.2.2 The requirements of Section 9.3.3.2.2.1 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.

COMMENTARY

9.3.3.2 *Beams* — This section applies to the design of lintels and beams.

9.3.3.2.2 *Longitudinal reinforcement*

9.3.3.2.2.1 The requirement that the nominal flexural strength of a beam not be less than 1.3 multiplied by the nominal cracking moment is imposed to prevent brittle failures. This situation may occur where a beam is so lightly reinforced that the bending moment required to cause yielding of the reinforcement is less than the bending moment required to cause cracking.

9.3.3.2.2.2 This exception provides sufficient additional reinforcement in members in which the amount of reinforcement required by Section 9.3.3.2.2.1 would be excessive.

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9.3.3.2.3 *Transverse reinforcement* — Transverse reinforcement shall be provided where V_u exceeds ϕV_{nm}. The strength level 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-leg stirrup with a 180-degree hook at each end.

(b) Transverse reinforcement shall be hooked around the longitudinal reinforcement.

(c) The minimum transverse reinforcement area divided by its spacing shall be at least 0.0007 *b*.

(d) The first stirrup 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 48 in. (1219 mm).

9.3.3.2.4 *Maximum Reinforcement* — The cross-sectional area of flexural tensile reinforcement shall not exceed the area required for the beam to be tension controlled as defined in Table 9.1.4.

COMMENTARY

9.3.3.2.3 *Transverse reinforcement* — Beams recognized in this section of the Code are often designed to resist only shear forces due to gravity loads. Beams that are controlled by high seismic forces and lateral drift should be designed as ductile members.

(a) Although some concerns have been raised regarding the difficulty in constructing beams containing a single-leg stirrup, the Committee feels such spacing limitations within beams inhibits the construction of necessary lap lengths required for two-leg stirrups. Furthermore, the added volume of reinforcement as a result of lap splicing stirrups may prevent adequate consolidation of the grout.

(b) The requirement that shear reinforcement be hooked around the longitudinal reinforcement not only facilitates construction but also confines the longitudinal reinforcement and contributes to the development of the shear reinforcement.

(c) A minimum area of transverse reinforcement is established to prevent brittle shear failures.

(d) Although different codes contain different spacing requirements for the placement of transverse reinforcement, the Committee has conservatively established this requirement.

(e) The requirements of this section establish limitations on the spacing and placement of reinforcement in order to increase member ductility.

9.3.3.2.4 *Maximum Reinforcement* — For Grade 60 reinforcement, the maximum flexural tensile reinforcement can be determined from:

$$\rho = \frac{A_s}{bd} = \frac{0.64 f'_m}{f_y}\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + 0.005}\right)$$

This corresponds approximately to 60% of the balanced reinforcement ratio.

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9.3.4 *Wall design for out-of-plane loads*

9.3.4.1 *Scope* — The requirements of Section 9.3.4 shall apply to the design of walls for out-of-plane loads.

9.3.4.2 *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 9-13 or Equation 9-14, as appropriate.

9.3.4.3 *Nominal shear strength* — The nominal shear strength shall be determined by Section9.3.3.1.2.

9.3.4.4 P-*delta effects*

9.3.4.4.1 Members shall be designed for the strength level 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 9.3.4.4.2 or Section9.3.4.4.3.

9.3.4.4.2 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 stress from the strength level axial load at the location of maximum moment satisfies the requirement calculated by Equation 9-20.

$$\left(\frac{P_u}{A_g}\right) \le 0.20 f'_m \qquad \text{(Equation 9-20)}$$

COMMENTARY

9.3.4 *Wall design for out-of-plane loads*

9.3.4.1 *Scope*

9.3.4.2 *Nominal axial and flexural strength* — When the depth of the equivalent stress block is in the face shell of a wall that is fully or partially grouted, the nominal moment may be found from:

$$M_n = \left(P_u/\phi + A_s f_y\right)\left(\frac{t_{sp}-a}{2}\right) + A_s f_y\left(d - \frac{t_{sp}}{2}\right)$$

$$a = \frac{A_s f_y + P_u/\phi}{0.80 f'_m b}$$

The above equations are valid for both centered and noncentered flexural reinforcement. For centered flexural reinforcement, $d = t_{sp}/2$ and the nominal moment, M_n, is obtained as:

$$M_n = \left(P_u/\phi + A_s f_y\right)\left(d - \frac{a}{2}\right)$$

These equations take into account the effect of compressive vertical loads increasing the flexural strength of the section. In the case of axial tension, the flexural strength is decreased.

9.3.4.4.2 The provisions of this section are derived from results of tests on simply supported specimens. Because the maximum bending moment and deflection occur near the mid-height of those specimens, this section includes only design equations for that condition. When actual conditions are not simple supports, the curvature of a wall under out-of-plane lateral loading will be different than that assumed by these equations. Using the principles of mechanics, the points of inflection can be determined and actual moments and deflections can be calculated under different support conditions. The designer should examine all moment and deflection conditions to locate the critical section using the assumptions outlined in Section9.3.4.

The criterion to limit vertical load on a cross section was included because the slender wall design method was based on data from testing with typical roof loads. For h/t ratios greater than 30, there is an additional limitation on the axial stress.

The required moment due to lateral loads, eccentricity of axial load, and lateral deformations is assumed maximum at mid-height of the wall. In certain design conditions, such as large eccentricities acting

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

The strength level moment and axial load shall be determined at the midheight of the wall and shall be used for design. The strength level moment, M_u, at the midheight of the wall shall be calculated using Equation 9-21.

$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u\delta_u \qquad \text{(Equation 9-21)}$$

Where:

$$P_u = P_{uw} + P_{uf} \qquad \text{(Equation 9-22)}$$

The deflection due to strength level loads (δ_u) shall be obtained using Equations 9-23 and 9-24.

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

$$\delta_u = \frac{5M_u h^2}{48E_m I_n} \qquad \text{(Equation 9-23)}$$

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

$$\delta_u = \frac{5M_{cr}h^2}{48E_m I_n} + \frac{5\left(M_u - M_{cr}\right)h^2}{48E_m I_{cr}} \qquad \text{(Equation 9-24)}$$

9.3.4.4.3 The strength level moment, M_u, shall be determined either by a second-order analysis, or by a first-order analysis and Equations 9-25 through 9-27.

$$M_u = \psi M_{u,0} \qquad \text{(Equation 9-25)}$$

Where $M_{u,0}$ is the strength level moment from first-order analysis.

$$\psi = \frac{1}{1-\frac{P_u}{P_e}} \qquad \text{(Equation 9-26)}$$

Where:

$$P_e = \frac{\pi^2 E_m I_{eff}}{h^2} \qquad \text{(Equation 9-27)}$$

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.

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simultaneously with small lateral loads, the design maximum moment may occur elsewhere. When this occurs, the designer should use the maximum moment at the critical section rather than the moment determined from Equation 9-21.

9.3.4.4.3 The moment magnifier provisions in this section were developed to provide an alternative to the traditional P-delta methods of Section 9.3.4.4.2. These provisions also allow other second-order analyses to be used.

The moment magnification equation is very similar to that used for slender wall design for reinforced concrete. Concrete design provisions use a factor of 0.75 in the denominator of the moment magnifier to account for uncertainties in the wall stiffness. This factor is retained for uncracked walls. It is not used for cracked walls. Instead, the cracked moment of inertia is conservatively used for the entire wall height. Trial designs indicated that using this approach matches design using Section 9.3.4.4.2. If a 0.75 factor were included along with using the cracked moment of inertia for the entire height would result in design moments approximately 7% greater than using Section 9.3.4.4.2. The Committee did not see any reason for the additional conservatism.

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9.3.4.4.4 The cracking moment of the wall shall be calculated using the modulus of rupture, f_r, taken from Table ~~9.1.9.2~~9.1.9.1.

9.3.4.4.5 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 fully grouted wall or a partially grouted wall with the neutral axis in the face shell shall be obtained from Equation 9-28 and Equation 9-29.

$$I_{cr} = nA_s\left(d-c\right)^2 + \frac{nP_u}{f_y}\left(\frac{t_{sp}}{2}-c\right)^2 + \frac{bc^3}{3} \quad \text{(Equation 9-28)}$$

$$c = \frac{A_s f_y + P_u}{0.64 f'_m\, b} \quad \text{(Equation 9-29)}$$

9.3.4.5 *Deflections* — The horizontal midheight deflection, δ_s, under allowable stress level loads shall be limited by the relation:

$$\delta_s \le 0.007\,h \quad \text{(Equation 9-30)}$$

P-delta effects shall be included in deflection calculation using either Section 9.3.4.5.1 or Section 9.3.4.5.2.

9.3.4.5.1 For simple support conditions top and bottom, the midheight deflection, δ_s, shall be calculated using either Equation 9-23 or Equation 9-24, as applicable, and replacing M_u with M_s and δ_u with δ_s.

COMMENTARY

9.3.4.4.4 The cracking moment, M_{cr}, is the calculated moment corresponding to first cracking. The Code permits the applied axial force to be included in the calculation of the cracking moment.

9.3.4.4.5 The Code requires that the neutral axis used to calculate the cracked moment of inertia be determined using the strain distribution at nominal strength. Amrhein and Lee (1984) used this condition to develop the original slender wall design provisions.

Equation 9-29 is similar to the equation presented in the commentary for Section 9.3.4.2. The commentary equation is used to determine the compression block depth, a, and should be 80% of the expression for c described in Equation 9-29. However, the expressions correctly address the applied axial load, P_u, differently because they do not describe the same conditions. Equation 9-28 is used to determine the cracked moment of inertia for a given load and thus the second order moment. The equation in Section 9.3.4.2 is used to determine the nominal moment capacity of a wall where the applied load, P_u, is set equal to the nominal axial capacity (the condition where $P_u = \phi P_n$), and thus P_u must be divided by ϕ.

9.3.4.5 *Deflections* — Historically, the recommendation has been to limit the deflection under allowable stress level loads to $0.01h$. The Committee has chosen a more stringent value of $0.007h$.

The Code limits the lateral deflection under allowable stress level loads. A wall loaded in this range returns to its original vertical position when the lateral load is removed, because the stress in the reinforcement is within its elastic limit.

9.3.4.5.1 Equation 9-23 is for mid-height deflection of a simply supported wall for an uncracked section, and Equation 9-24 is for mid-height deflection for a cracked section. A wall is assumed to deflect as an uncracked section until the modulus of rupture is reached, after which it is assumed to deflect as a cracked section. The cracked moment of inertia is conservatively assumed to apply over the entire height of the wall. The cracked moment of inertia, I_{cr}, for a fully grouted or partially grouted cross section is usually the same as that for a hollow section because the compression stress block is generally within the thickness of the face shell.

These equations represent good approximations to test results, assuming that the wall is simply supported top and bottom, and is subjected to a uniformly distributed lateral load. If the wall is fixed at top, bottom, or both, other formulas should be developed considering the support conditions at the top or bottom and considering the possible deflection or rotation of the foundation, roof, or floor diaphragm.

The Code requires that the neutral axis used to calculate the cracked moment of inertia be determined using the strain distribution at nominal strength. Amrhein and Lee (1984) used this condition to develop the original slender wall design provisions.

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9.3.4.5.2 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 9-27.

9.3.5 *Wall design for in-plane loads*

9.3.5.1 *Scope* — The requirements of Section 9.3.5 shall apply to the design of walls to resist in-plane loads.

9.3.5.2 *Reinforcement* — 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 8 ft (2.44 m).

9.3.5.3 *Flexural and axial strength* — The nominal flexural and axial strength shall be determined in accordance with Section 9.3.3.1.1.

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

9.3.5.5 *Shear-Friction strength* — Provisions of this section shall apply to shear transfer across horizontal interfaces. The nominal shear-friction strength, V_{nf}, at a horizontal interface shall be determined as follows.

Where $M_u/(V_u d_v) \leq 0.5$

$$V_{nf} = \mu\left(A_{sp} f_y + P_u\right) \geq 0 \qquad \text{(Equation 9-31)}$$

The reinforcement A_{sp} in Equation 9-31 shall be adequately anchored above and below the horizontal shear plane to develop the yield strength of the reinforcement. The value of P_u is negative when it is a tension force. The coefficient of friction, μ, shall be 1.0 for masonry on concrete with an unfinished surface, or masonry on concrete with a finished surface that has been intentionally roughened; μ shall be 0.70 for all other situations.

Where $M_u/(V_u d_v) \geq 1.0$

$$V_{nf} = 0.65\left(0.75 A_{sp} f_y + P_u\right) \geq 0 \qquad \text{(Equation 9-32)}$$

COMMENTARY

Equation 9-28 and 9-29 are valid for both centered and non-centered vertical reinforcement. The modification term of $(t_{sp}/2d)$ in Equation 9-28 accounts for a reduction in the contribution of the axial load to the cracked moment of inertia when the reinforcement is near the face of the wall.

9.3.4.5.2 This section allows other second-order analyses to be used to predict wall deflections, including first-order deflections amplified using a moment magnification factor.

Less conservative estimation for first-order wall deformation can be obtained using an effective I value that accounts for partial cracking of the sections, such as that described in Section 5.~~2~~3.1.5.2.

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9.3.5 *Wall design for in-plane loads*

9.3.5.5 *Shear-Friction strength* — When subjected to in-plane lateral loads, walls that have a low axial compressive load and a low shear-span ratio are vulnerable to shear sliding, which normally occurs at the base. Shear sliding is resisted by three mechanisms, namely, the friction, the dowel action of the reinforcement crossing the shear plane, and the shear strength of the reinforcement. The dowel action and shear strength of the reinforcement will not be fully activated until the friction resistance has been overcome and shear sliding initiates. Shear sliding can cause severe damage to the masonry due to the simultaneous actions of the shear stress, compressive stress, and dowel action; it can weaken lap splices adjacent to the shear plane; it can fracture the reinforcement crossing the shear plane.

The coefficient of friction is a function of the roughness of the surface. See Commentary Section 8.3.6 for additional information.

Equation 9-31 is adopted from FEMA 306 (ATC 1998). It has been found to be adequate for walls with low shear-span ratios. However, studies by Murcia-Delso and Shing (2012) have concluded that Equation 9-31 over-predicts the shear-friction resistance of walls with $M_u/(V_u d_v) = 1.0$. Experimental data of Shing et al (1989) and Ahmadi et al (2013) have shown that flexure-

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Where $M_u/(V_u d_v)$ is between 0.5 and 1.0, the value of V_{nf} shall be determined by linear interpolation between the values given by Equations 9-31 and 9-32.

9.3.5.6 *Ductility requirements* — Intermediate and special reinforced masonry shear walls shall either:

a) comply with the maximum reinforcement requirements of Section 9.3.5.6.1, or

b) comply with alternate ductility provisions of Section 9.3.5.6.2.

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dominated walls normally reached their flexural capacities before significant base sliding occurred. When a wall reaches its flexural capacity, a portion of the vertical reinforcement crossing the horizontal shear plane can be in compression. The reinforcement in compression reduces the compressive force exerted by masonry on the sliding surface and thus decreases the friction resistance, while the reinforcement in tension provides a clamping force that enhances the friction resistance.

Equation 9-32 accounts for this effect and also for the reduction of the coefficient of friction as the masonry in compression is damaged when the flexural capacity is reached. The equations have been validated by the wall test data of Shing et al (1989), Voon and Ingham (2006), and Ahmadi et al (2013).

In Equations 9-31 and 9-32, A_{sp} represents the reinforcement crossing the net shear area (A_{nv}), which is limited to the webs of flanged walls.

9.3.5.6 *Ductility requirements* — The maximum reinforcement requirements of Section 9.3.5.6.1 are intended to ensure that an intermediate or a special reinforced masonry shear wall has sufficient inelastic deformation capacity under the design-basis earthquake of ASCE/SEI 7 or the model building codes. Inelastic deformability is the ability of a structure or structural member to continue to sustain gravity loads as it deforms laterally under earthquake (or some other type of) excitation beyond the stage where the response of the structure or the structural member to that excitation is elastic (that is, associated with no residual displacement or damage). As an alternative to the maximum reinforcement provision of Section 9.3.5.6.1, Section 9.3.5.6.2 may be used to demonstrate that the wall has adequate ductility for the imposed demands, or to enhance the wall ductility by means of specially confined boundary elements.

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9.3.5.6.1 *Maximum area of flexural tensile reinforcement* — The cross-sectional area of flexural tensile reinforcement shall not exceed the area required to maintain axial equilibrium under in-plane loads under the following conditions:

(a) A strain gradient shall be assumed, corresponding to a strain in the extreme tensile reinforcement equal to the value given in Table 9.3.5.6.1 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.

Table 9.3.5.6.1: Strain in Extreme Tensile Reinforcement

Shear Wall	Tensile strain in reinforcement	
	$M_u/(V_u d_v) < 1$	$M_u/(V_u d_v) \geq 1$
Intermediate reinforced	$1.5\varepsilon_y$	$3.0\varepsilon_y$
Special reinforced	$1.5\varepsilon_y$	$4.0\varepsilon_y$

COMMENTARY

9.3.5.6.1 *Maximum area of flexural tensile reinforcement* — Prior to the 2022 edition of this Code, maximum reinforcement provisions applied to ordinary reinforced shear walls, and to walls loaded in the out-of-plane direction. With the adoption of tension and compression-controlled sections in the 2022 Code, the maximum reinforcement requirements for these walls are waived because of the added conservatism associated with compression-controlled sections. Although strength reductions are not directly exchangeable with ductility, this waiver was deemed appropriate since inelastic curvature for these wall types is relatively small per the former maximum reinforcement requirements. The decrease in strength-reduction factor for compression-controlled sections adds an additional margin of safety which compensates for a small reduction in ductility attributable to possible increases in reinforcement amounts exceeding the former limit.

Flexural tensile reinforcement is limited to a maximum amount to ensure that masonry compressive strains will not exceed ultimate values. In other words, the compressive zone of the member will not crush before the tensile reinforcement develops the inelastic strain consistent with the curvature ductility implied by the R value used in design.

The maximum reinforcement in intermediate and special walls is limited in accordance with a prescribed strain distribution based on a tensile strain equal to a factor times the yield strain for the reinforcement closest to the edge of the member, and a maximum masonry compressive strain equal to 0.0025 for concrete masonry or 0.0035 for clay-unit masonry. By limiting longitudinal reinforcement in this manner, inelastic curvature capacity is directly related to the strain gradient.

The tensile strain factor varies in accordance with the amount of curvature ductility expected, and ranges from 1.5 to 4 for specially reinforced masonry shear walls. Expected curvature ductility, controlled by the factor on tensile yield strain, is assumed to be associated directly with the displacement ductility, or the value of C_d as given for the type of component. For example, a strain factor of 3 for intermediate reinforced masonry shear walls corresponds to the slightly smaller C_d factor of 2.5, and a strain factor of 4 for specially reinforced walls corresponds to the slightly smaller C_d factor of 3.5.

The maximum reinforcement is determined by considering the prescribed strain distribution, determining the corresponding stress and force distribution, and using statics to sum axial forces.

For a fully grouted shear wall subjected to in-plane loads with uniformly distributed reinforcement, the maximum area of reinforcement per unit length of wall is determined as:

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9.3.5.6.2 *Alternate ~~A~~approaches to ~~W~~wall ~~D~~ductility*

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

1. $P_u \leq 0.10\, A_n f'_m$ for geometrically symmetrical wall sections

   $P_u \leq 0.05\, A_n f'_m$ for geometrically unsymmetrical wall sections; and either

2. $\dfrac{M_u}{V_u d_v} \leq 1.0$

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$$\frac{A_s}{d_v} = \frac{0.64 f'_m\, b\left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \dfrac{P}{d_v}}{f_y\left(\dfrac{\alpha\varepsilon_y - \varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right)}$$

For a fully grouted member with only concentrated tension reinforcement, the maximum reinforcement is:

$$\rho = \frac{A_s}{bd} = \frac{0.64 f'_m\left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \dfrac{P}{bd}}{f_y}$$

If there is concentrated compression reinforcement with an area equal to the concentrated tension reinforcement, A_s , the maximum reinforcement is:

$$\rho = \frac{A_s}{bd} = \frac{0.64 f'_m\left(\dfrac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \dfrac{P}{bd}}{f_y - \min\left\{\varepsilon_{mu} - \dfrac{d'}{d}\left(\varepsilon_{mu} + \alpha\varepsilon_y\right), \varepsilon_y\right\} E_s}$$

where d' is the distance from the extreme compression fiber to the centroid of the compression reinforcement.

Because axial force is implicitly considered in the determination of maximum longitudinal reinforcement, inelastic curvature capacity can be relied on no matter what the level of axial compressive force. Also, confinement reinforcement is not required because the maximum masonry compressive strain will be less than ultimate values.

The axial force is the expected load at the time of the design earthquake. It is derived from ASCE/SEI 7 Allowable Stress Load Combination 9 and consideration of the horizontal component of the seismic loading. The vertical component of the earthquake load, E_v, should not be included in calculating the axial force for purposes of determining maximum area of flexural tensile reinforcement.

9.3.5.6.2 *Alternate ~~A~~approaches to ~~W~~wall ~~D~~ductility*

9.3.5.6.2.1 This subsection sets up some "screens" with the expectation that many, if not most, shear walls will go through the screens, in which case no special boundary elements would be required. This situation will be the case when a shear wall is lightly axially loaded and it is either short or is moderate in height and is subject to only moderate shear stresses.

The threshold values are adapted from the design procedure for special reinforced concrete shear walls in the 1997 Uniform Building Code (UBC). In the early 1990s, when this procedure of the 1997 UBC was first being developed, an ad hoc subcommittee within the Seismology Committee of the Structural Engineers Association of California had limited, unpublished parametric studies done,

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or

3. $V_u \leq 3A_{nv}\sqrt{f'_m}$ and $\dfrac{M_u}{V_u d_v} \leq 3.0$

9.3.5.6.2.2 The need for special boundary elements at the edges of shear walls shall be evaluated in accordance with Section 9.3.5.6.2.3 or 9.3.5.6.2.4. The requirements of Section 9.3.5.6.2.5 shall also be satisfied.

9.3.5.6.2.3 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 9.3.5.6.2.4

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

$$c \geq \frac{l_w}{600\left(1.5C_d\delta_{ne} / h_w\right)}$$

and c is calculated for the P_u given by ASCE/SEI 7 Strength Design Load Combination 6 ($1.2D + E_v + E_h + L +$ ~~0.2~~$0.15S$) or the corresponding strength design load combination of the legally adopted building code, and the corresponding nominal moment strength, M_n, at the base critical section. The load factor on L in Combination 6 is reducible to 0.5, as per exceptions to Section 2.3.6 of ASCE/SEI 7.

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showing that a reinforced concrete shear wall passing through the "screens" could not develop sufficiently high compressive strains in the concrete to warrant special confinement. In the case of masonry, strains requiring special confinement would be values exceeding the maximum usable strains of Section 9.3.2 (c).

9.3.5.6.2.2 Two approaches for evaluating detailing requirements at wall boundaries are included in Section 9.3.5.6.2. Section 9.3.5.6.2.3 allows the use of displacement-based design of walls, in which the structural details are determined directly on the basis of the expected lateral displacements of the wall under the design-basis earthquake. The provisions of Section 9.3.5.6.2.4 are conservative for assessing required transverse reinforcement at wall boundaries for many walls. The requirements of Section 9.3.5.6.2.5 apply to shear walls designed by either Section 9.3.5.6.2.3 or 9.3.5.6.2.4.

9.3.5.6.2.3 Section 9.3.5.6.2.3 is based on the assumption that inelastic response of the wall is dominated by flexural action at a critical, yielding section – typically at the base. The wall should be proportioned so that the critical section occurs where intended (at the base).

(a) The following explanation, including Figure CC-9.3-2, is adapted from a paper by Wallace and Orakcal (2002). The relationship between the wall top displacement and wall curvature for a wall of uniform cross-section with a single critical section at the base is presented in Figure CC-9.3-2. The provisions of this Code are based on a simplified version of the model presented in Figure CC-9.3-2(a). The simplified model, shown in Figure CC-9.3-2(b), neglects the contribution of elastic deformations to the top displacement, and moves the center of the plastic hinge to the base of the wall. Based on the model of Figure CC-9.3-2, the relationship between the top displacement and the curvature at the base of the wall is:

$$\cancel{1.5C_d\delta_{ne} = \theta_p h_w = (\phi_u \ell_p)h_w = \left(\phi_u \frac{\ell_w}{2}\right)h_w}$$

$$\delta_{MCE} = \theta_p h_w = (\phi_u \ell_p)h_w = \left(\phi_u \frac{\ell_w}{2}\right)h_w$$

(Equation 1)

assuming that $\ell_p = \ell_w / 2$, as is permitted to be assumed by the 1997 UBC,

where ~~φ_u~~ ϕ_u = ultimate curvature, and

θ_p = plastic rotation at the base of the wall.

~~The 1.5 factor in front of the term $C_d\delta_{ne}$ amplifies the displacement so that it corresponds to the displacement that would be expected for the Risk-Targeted Maximum Considered Earthquake (MCE$_R$) event. This is done~~ The displacement associated with the Risk-

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Targeted Maximum Considered Earthquake (MCER) is used to align the detailing requirements with the intent of the building code to have a low probability of collapse in the MCE_R event.

If at the stage where the top deflection of the wall is ~~δ_{ne},~~ δ_{MCE}, the extreme fiber compressive strain at the critical section at the base does not exceed ε_{mu}, no special confinement would be required anywhere in the wall. Figure CC-9.3-3 illustrates such a strain distribution at the critical section. The neutral axis depth corresponding to this strain distribution is c_{cr}, and the corresponding ultimate curvature is $\phi_u = \varepsilon_{mu} / c_{cr}$. From Equation 1,

$$\cancel{1.5C_d\delta_{ne}} = \left(\frac{\varepsilon_{mu}}{c_{cr}}\frac{\ell_w}{2}\right)\cancel{h_w}\ \delta_{MCE} = \left(\frac{\varepsilon_{mu}}{c_{cr}}\frac{\ell_w}{2}\right)h_w$$

(Equation 2a)

or, $\cancel{c_{cr}} = \dfrac{\varepsilon_{mu}}{2}\dfrac{\ell_w}{(1.5C_d\delta_{ne}/h_w)}$ $c_{cr} = \dfrac{\varepsilon_{mu}}{2}\dfrac{\ell_w}{(\delta_{MCE}/h_w)}$

(Equation 2b)

From the equations above (see Figure CC-9.3-3), special detailing would be required if:

$$c \geq \frac{\varepsilon_{mu}}{2}\frac{\ell_w}{(1.5C_d\delta_{ne}/h_w)} = \frac{0.003}{2}\frac{\ell_w}{(1.5C_d\delta_{ne}/h_w)}$$

$$= \frac{\ell_w}{667\,(1.5C_d\delta_{ne}/h_w)} \approx \frac{\ell_w}{600\,(1.5C_d\delta_{ne}/h_w)}$$

$$c \geq \frac{\varepsilon_{mu}}{2}\frac{\ell_w}{(\delta_{MCE}/h_w)} = \frac{0.003}{2}\frac{\ell_w}{(\delta_{MCE}/h_w)}$$

$$= \frac{\ell_w}{667\,(\delta_{MCE}/h_w)} \approx \frac{\ell_w}{600\,(\delta_{MCE}/h_w)}$$

because if the neutral axis depth exceeded the critical value, the extreme fiber compressive strain would exceed the maximum usable strain ε_{mu}. For purposes of this derivation, and to avoid having separate sets of drift-related requirements for clay and concrete masonry, a single useful strain of 0.003 is used, representing an average of the design values of 0.0025 for concrete masonry and 0.0035 for clay masonry.

(b) Where special boundary elements are required by Section 9.3.5.6.2.3 (a), the special boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of ℓ_w or $M_u/4V_u$.

(b) These special extensions are intended to be an upper-bound estimate of the plastic hinge length for special reinforced masonry shear walls.

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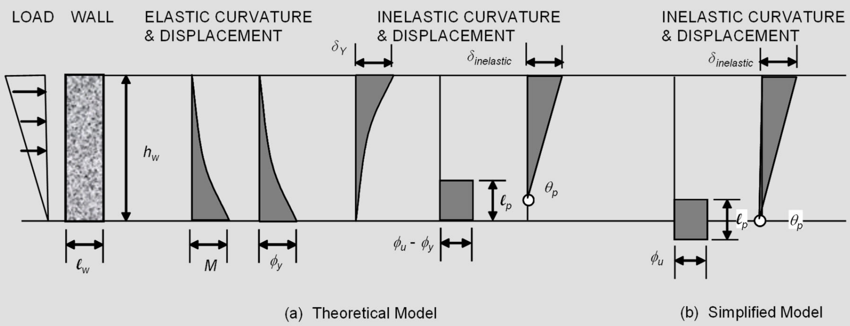

Figure CC-9.3-2 — Wall curvature and displacement

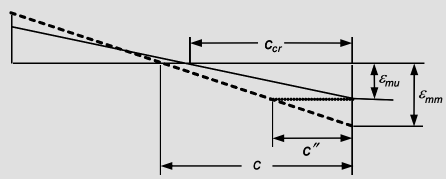

Figure CC-9.3-3 —Strain distribution at critical section

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9.3.5.6.2.4 Shear walls not designed by Section 9.3.5.6.2.3 shall have special boundary elements at boundaries and edges around openings in shear walls where the maximum extreme fiber compressive stress, corresponding to forces from strength level loads including earthquake effect, exceeds $0.2 f'_m$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15 f'_m$. Stresses shall be calculated from strength level loads using a linearly elastic model and net section properties. For walls with flanges, an effective flange width as defined in Section ~~5.1.1.1.3~~5.2.3.3 shall be used.

9.3.5.6.2.5 Where special boundary elements are required by Section 9.3.5.6.2.3 or 9.3.5.6.2.4, requirements (a) through (d) in this section shall be satisfied and tests shall be performed to verify the strain capacity of the element:

COMMENTARY

9.3.5.6.2.4 A stress-based approach is included to address wall configurations to which the application of displacement-based approach is not appropriate (for example, walls with openings, walls with setbacks, walls not controlled by flexure).

This Code has adopted the stress-based triggers of ACI 318-99 for cases where the displacement-based approach is not applicable, simply changing the threshold values of $0.2 f'_c$ and $0.15 f'_c$ for reinforced concrete walls to $0.2 f'_m$ and $0.15 f'_m$, respectively, for reinforced masonry walls. Other aspects of the ACI 318-99 approach are retained. Design for flexure and axial loads does not change depending on whether the neutral axis-based trigger or the stress-based trigger is used.

9.3.5.6.2.5 This Code requires that testing be done to verify that the detailing provided is capable of developing a strain capacity in the boundary element that would be in excess of the maximum imposed strain. Reasonably extensive tests need to be conducted to develop prescriptive detailing requirements for specially confined boundary elements of intermediate as well as special reinforced masonry shear walls.

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(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.

COMMENTARY

(a) Figure CC-9.3-3 shows that when the neutral axis depth c exceeds the critical neutral axis depth c_{cr}, the extreme compression fiber strain in the masonry reaches a value ε_{mm} in excess of the maximum usable strain ε_{mu}. The corresponding ultimate curvature ϕ is ε_{mu} / c. Based on the model of Figure CC-9.3-2(b), with ℓ_p = ℓ_w/2 and assuming the wall experiences the Risk-Targeted Maximum Considered Earthquake (MCE_R) event:

$$\cancel{1.5C_d\delta_{ne} = \theta_p h_w = (\phi_u \ell_p)h_w = \left(\frac{\varepsilon_{mm}}{c}\frac{\ell_w}{2}\right)h_w}$$

$$\delta_{MCE} = \theta_p h_w = (\phi_u \ell_p)h_w = \left(\frac{\varepsilon_{mm}}{c}\frac{\ell_w}{2}\right)h_w \qquad \text{(Equation 3)}$$

From Equation 3:

$$\cancel{\varepsilon_{mm} = 2\left(\frac{1.5C_d\delta_{ne}}{h_w}\right)\left(\frac{c}{\ell_w}\right)}$$

$$\varepsilon_{mm} = 2\left(\frac{\delta_{MCE}}{h_w}\right)\left(\frac{c}{\ell_w}\right) \qquad \text{(Equation 4)}$$

The wall length over which the strains exceed the limiting value of ε_{mu}, denoted as c'', can be determined using similar triangles from Figure CC-9.3-3:

$$c'' = c\left(1 - \frac{\varepsilon_{mu}}{\varepsilon_{mm}}\right) \qquad \text{(Equation 5)}$$

An expression for the required length of confinement can be developed by combining Equations 2 and 3:

$$\cancel{\frac{c''}{\ell_w} = \frac{c}{\ell_w} - \frac{(\varepsilon_{mu}/2)}{(1.5C_d\delta_{ne}/h_w)}}$$

$$\frac{c''}{\ell_w} = \frac{c}{\ell_w} - \frac{(\varepsilon_{mu}/2)}{(\delta_{MCE}/h_w)} \qquad \text{(Equation 6)}$$

The term c/ℓ_w in Equation 6 accounts for the influence of material properties (f'_m, f_y), axial load, geometry, and quantities and distribution of reinforcement, whereas the term $\cancel{(\varepsilon_{mu}/2)/(1.5C_d\delta_{ne}/h_w)}$ $(\varepsilon_{mu}/2)/(\delta_{MCE}/h_w)$ accounts for the influence of system response (roof displacement) and the maximum usable strain of masonry.

The wall length over which special transverse reinforcement is to be provided is based on Equation 6, with a value of $\cancel{1.5C_d\delta_{ne}/h_w}$ δ_{MCE}/h_w = 1.5(0.01) = 0.015:

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COMMENTARY

$$\frac{c''}{\ell_w} = \frac{c}{\ell_w} - \frac{(0.003/2)}{0.015} = \frac{c}{\ell_w} - 0.1 \geq \frac{c}{2} \quad \text{(Equation 7)}$$

The value of ($\cancel{1.5C_d\delta_{ne}/h_w}$ δ_{MCE}/h_w) = 0.015 was selected to provide an upper-bound estimate of the mean drift ratio of typical masonry shear wall buildings designed in accordance with ASCE/SEI 7, based on a maximum permitted drift of 0.01 in the design earthquake, amplified by a 1.5 factor for the MCE_R event. Thus, the length of the wall that must be confined is conservative for many buildings. The value of $c/2$ represents a minimum length of confinement, is adopted from ACI 318-99, and is arbitrary.

(b) In flanged sections, the special boundary element shall include the effective flange width in compression and shall extend at least 12 in. (305 mm) into the web.

(b) This requirement originated in the 1997 UBC. Where flanges are highly stressed in compression, the web-to-flange interface is likely to be highly stressed and may sustain local crushing failure unless special boundary element reinforcement extends 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 12 in. (305 mm) into the footing or mat.

(c) The same extension is required for special boundary element transverse reinforcement in special reinforced concrete shear walls and for special transverse reinforcement in reinforced concrete columns supporting reactions from discontinued stiff members in buildings assigned to high seismic design categories.

(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.

(d) Because horizontal reinforcement is likely to act as web reinforcement in walls requiring boundary elements, it needs to be fully anchored in boundary elements that act as flanges. Achievement of this anchorage is difficult when large transverse cracks occur in the boundary elements. Standard 90-degree hooks or mechanical anchorage schemes, instead of a straight development length are recommended.

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CHAPTER 10
PRESTRESSED MASONRY

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10.1 — General

10.1.1 *Scope*

This chapter provides requirements for design of masonry members that are prestressed with bonded or unbonded prestressing tendons.

10.1.2 The wall provisions of Part 1, Part 2, and Section 8.1 shall apply to prestressed masonry walls except as noted in this Chapter.

10.1.3 Masonry in walls shall be laid in running bond unless a bond beam or other technique is used to distribute anchorage forces.

10.1.4 For masonry ~~members~~walls, beams, and lintels, the prestressing force shall be added to load combinations, except as modified by Section 10.4.2.

COMMENTARY

10.1 — General

10.1.1 *Scope*

Prestressing forces are used in masonry members to reduce or eliminate tensile stresses due to externally applied loads by using controlled precompression. The precompression is generated by prestressing tendons, either bars, wires, or strands, that are contained in openings in the masonry, which may be grouted. The prestressing tendons can be pre-tensioned (stressed against external abutments prior to placing the masonry), or post-tensioned (stressed against the masonry after it has been placed). Provisions for columns are not included in this chapter but will be developed in future editions of the Code.

Most construction applications to date have involved post-tensioned masonry for its ease of construction and overall economy. Consequently, these code provisions primarily focus on post-tensioned masonry. Although not very common, pre-tensioning has been used to construct prefabricated masonry panels. A more detailed review of prestressed masonry systems and applications is given in Schultz and Scolforo (1991).

Throughout this Code and TMS 602, references to "reinforcement" apply to non-prestressed reinforcement. These references do not apply to prestressing tendons, except as explicitly noted in Chapter 10. Requirements for prestressing tendons use the terms "prestressing tendon" or "tendon." The provisions of Chapter 10 do not require a mandatory quantity of reinforcement or bonded prestressing tendons for prestressed masonry members.

Anchorage forces are distributed within a member similar to the way in which concentrated loads are distributed (as described in Section 5.1.~~3~~1; see Figure CC-5.1-~~5~~1). However, research (Woodham and Hamilton (2003)) has indicated that for walls prestress losses can distribute to adjacent tendons as far laterally from the anchorage as the height of the wall.

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10.1.5 Masonry beams and lintels shall have a uniform width and be fully grouted or solid, and reinforced to distribute anchorage forces. If a masonry beam or lintel is integral within a wall, only the beam or lintel portion need be fully grouted or solid.

10.2 — Design methods

10.2.1 *General*
Members shall be designed to meet the strength provisions in this Chapter and checked for allowable stress level load requirements. The provisions of Section 10.4.3 shall apply for the calculation of nominal moment strength. Loading and load combinations shall be in accordance with the provisions of Sections 4.1.2, except as noted in this Chapter.

10.2.2 *After transfer*
Immediately after the transfer of prestressing force to the masonry, limitations on masonry stresses given in this chapter shall be based upon f'_{mi}.

10.3 — Permissible stresses in prestressing tendons

10.3.1 *Jacking force*
The stress in prestressing tendons due to the jacking force shall not exceed $0.94 f_{py}$, nor $0.80 f_{pu}$, nor the maximum value recommended by the manufacturer of the prestressing tendons or anchorages.

10.3.2 *Immediately after transfer*

COMMENTARY

10.1.5 While masonry walls are to be laid in running bond, masonry beams and lintels are effectively not laid in running bond relative to the horizontal post-tensioning tendon. Therefore, the ends of masonry beams and lintels must be detailed to distribute the anchorage of the tendons into the masonry.

10.2 — Design methods

Originally, prestressed masonry was designed using allowable stress design with a moment strength check for members with laterally restrained tendons. The British code for prestressed masonry (BSI (1985); Phipps (1992)) and extensive research on the behavior of prestressed masonry were considered. Summaries of prestressed masonry research and proposed design criteria are available in the literature (Schultz and Scolforo (1992 and 1992b); Ganz, (1990); Curtin et al (1988); Phipps and Montague (1976)). Design methods are now based upon strength provisions with serviceability checks based upon allowable stress level requirements.

A masonry wall is typically prestressed prior to 28 days after construction, sometimes within 24 hours after construction. The specified compressive strength of the masonry at the time of prestressing (f'_{mi}) is used to determine allowable prestressing levels. This strength will likely be a fraction of the 28-day specified compressive strength. Assessment of masonry compressive strength immediately before the transfer of prestress should be by testing of masonry prisms. ~~or by a record of strength gain over time of masonry prisms constructed of similar masonry units, mortar, and grout, when subjected to similar curing conditions.~~ If concrete end blocks are used, assessment of concrete compressive strength immediately before the transfer of prestress should be done according to concrete design specifications (ACI 318).

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10.3 — Permissible stresses in prestressing tendons

Allowable prestressing-tendon stresses are based on criteria established for prestressed concrete (ACI 318-19~~, 2011~~). Allowable prestressing-tendon stresses are for jacking forces and for the state of stress in the prestressing tendon immediately after the prestressing has been applied, or transferred, to the masonry. When calculating the prestressing-tendon stress immediately after transfer of prestress, consider all sources of short term prestress losses. These sources include such items as anchorage seating loss, elastic shortening of masonry, and friction losses.

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The stress in the prestressing tendons immediately after transfer of the prestressing force to the masonry shall not exceed 0.82 f_{py} nor 0.74 f_{pu} .

10.3.3 *Post-tensioned masonry members*

At the time of application of prestress, the stress in prestressing tendons at anchorages and couplers shall not exceed 0.78 f_{py} nor 0.70 f_{pu} .

10.3.4 *Effective prestress*

The calculated effective stress in the prestressing tendons under allowable stress level loads, f_{se} , shall include the effects of the following:

(a) anchorage seating losses,

(b) elastic shortening of masonry,

(c) creep of masonry,

(d) shrinkage of concrete masonry,

(e) relaxation of prestressing tendon stress,

(f) friction losses,

(g) irreversible moisture expansion of clay masonry, and

(h) thermal effects.

COMMENTARY

10.3.4 *Effective prestress*

The state of stress in a prestressed masonry member must be checked for each stage of loading. For each loading condition, the effective prestress, f_{se}, should be used in the calculation of stresses and member strength. Effective prestress is not a fixed quantity over time. Research on the loss and gain of prestress in prestressed masonry is extensive and includes testing of time-dependent phenomena such as creep, shrinkage, moisture expansion, and prestressing-tendon stress relaxation (PCI (1975); Lenczner (1985); Lenczner (1987); Shrive (1988)).

Instantaneous deformation of masonry due to the application of prestress may be calculated by the modulus of elasticity of masonry given in Section 4.2.2. Creep, shrinkage, and moisture expansion of masonry may be calculated by the coefficients given in Section 4.2. Change in effective prestress due to elastic deformation, creep, shrinkage, and moisture expansion should be based on relative modulus of elasticity of masonry and prestressing steel.

The stressing operation and relative placement of prestressing tendons should be considered in calculating losses. Elastic shortening during post-tensioning can reduce the stress in adjacent tendons that have already been stressed. Consequently, elastic shortening of the wall should be calculated considering the incremental application of post-tensioning. That elastic shortening should then be used to estimate the total loss of prestress. Alternatively, post-tensioning tendons can be prestressed to compensate for the elastic shortening caused by the incremental stressing operation.

Prestressing steel that is tensioned to a large fraction of its yield strength and held at a constant strain will relax, requiring less stress to maintain a constant strain. The phenomenon of stress relaxation is associated with plastic deformation and its magnitude increases with steel stress as a fraction of steel strength. Prestressing steels that conform to ASTM A416/A416M (~~2012~~2018), A421/A421M (~~2010~~2021), and A722/A722 (~~2007~~2018) are stabilized for low relaxation losses during production. Other steel types that do not have this stabilization treatment may exhibit considerably higher relaxation losses. Their relaxation losses must be carefully assessed by testing. The loss of effective prestress due to stress relaxation of the prestressing tendon is dependent upon the level of prestress, which changes with time-dependent phenomena such as creep, shrinkage, and moisture expansion of the masonry. An appropriate formula

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for predicting prestress loss due to relaxation has been developed (Lenczner (1985); Lenczner (1987); Shrive (1988)). Alternately, direct addition of the steel stress-relaxation value provided by the manufacturer can be used to calculate prestress losses and gains.

Friction losses are minimal or nonexistent for most post-tensioned masonry applications, because prestressing tendons are usually straight and contained in cavities. For anchorage losses, manufacturers' information should be used to calculate prestress losses. Changes in prestress due to thermal fluctuations may be neglected if masonry is prestressed with high-strength prestressing steels. Loss of prestressing should be calculated for each design to determine effective prestress. Calculations should be based on the particular construction materials and methods as well as the climate and environmental conditions. Committee experience, research, and field experience with post-tensioned wall designs from Switzerland, Great Britain, Australia, and New Zealand have indicated that prestress losses are expected to be in the following ranges (Woodham and Hamilton (2003); Hamilton and Badger (2000); Biggs and Ganz (1998); NCMA TEK 14-20A (2002)):

(a) Initial loss after jacking –5% to 10%

(b) Total losses after long-term service for concrete masonry – 30% to 35%

(c) Total losses after long-term service for clay masonry – 20% to 25%

The values in (b) and (c) include both the short-term and long-term losses expected for post-tensioning. The Committee believes these ranges provide reasonable estimates for typical wall applications to date, unless calculations, experience, or construction techniques indicate different losses are expected. To date, prestressed walls have been built using ASTM C90 block with relatively small amounts of intermediate grade reinforcement, such as A722 Grade 120 bar, which is prestressed to magnitudes near the maximum limits permitted by code. Losses for prestressing steels with a yield strength below 100 ksi (690 MPa) can be considerably higher than the ranges suggested above (TMS Responds, 2021).

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10.4 — Axial compression and flexure

10.4.1 *General*

10.4.1.1 Members subjected to axial compression, flexure, or to combined axial compression and flexure shall be designed according to the provisions of Section 8.2.4, except as noted in Section 10.4.1.2, 10.4.1.3, 10.4.2, and 10.4.3.

10.4.1.2 The allowable compressive stresses due to axial loads, F_a, and flexure, F_b, and the allowable axial force in Equation 8-10 shall be permitted to be increased by 20 percent for the stress condition immediately after transfer of prestress.

10.4 — Axial compression and flexure

10.4.1 *General*

The requirements for prestressed masonry members subjected to axial compression and flexure are separated into those with laterally unrestrained prestressing tendons and those with laterally restrained prestressing tendons. This separation was necessary because the flexural behavior of a member depends significantly upon the lateral restraint of the prestressing tendon. Lateral restraint of a prestressing tendon is typically provided by grouting the cell or void containing the tendon before or after transfer of prestressing force to the masonry. Lateral restraint may be provided by placing the masonry in contact with the tendon or the protective

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10.4.1.3 Masonry shall not be subjected to flexural tensile stress from the combination of prestressing force and dead load.

10.4.2 *Allowable stress level load requirements*

10.4.2.1 For members with laterally unrestrained prestressing tendons, the prestressing force, P_{ps}, shall be considered for the calculation of the axial load, P, in Equation 8-10.

10.4.2.2 For members with laterally restrained prestressing tendons that are concentric, the prestressing force, P_{ps}, shall not be considered for the calculation of the axial load, P, in Equation 8-10. For members with laterally restrained prestressing tendons that are not concentric, the prestressing force, P_{ps}, shall be considered for the calculation of the axial load, P, in Equation 8-10.

COMMENTARY

sheathing of the tendon at periodic intervals along the length of the prestressing tendon.

Allowable compressive stresses for prestressed masonry address two distinct loading stages; stresses immediately after transfer of prestressing force to the masonry member and stresses after all prestress losses and gains have taken place. The magnitude of allowable axial compressive stress and bending compressive stress after all prestress losses and gains are consistent with those for unreinforced masonry in Section 8.2. Immediately after transfer of prestressing, allowable compressive stresses and applied axial load should be based upon f'_{mi} and may be increased by 20 percent. This means that the factors of safety at the time of the transfer of prestress may be lower than those after prestress losses and gains occur. The first reason for this is that the effective precompression stress at the time of transfer of prestressing almost certainly decreases over time and masonry compressive strength most likely increases over time. Second, loads at the time of transfer of prestressing, namely prestress force and dead loads, are known more precisely than loads throughout the remainder of service life.

Cracking of prestressed masonry under permanent loads is to be avoided. The prestressing force and the dead weight of the member are permanent loads. Cracking under permanent loading conditions is not desirable due to the potential for significant water penetration, which may precipitate corrosion of the prestressing tendons and accessories and damage to interior finishes. Masonry provides a significant flexural tensile resistance to cracking, as reflected by the allowable flexural tensile stress values stated in Section 8.2. Consequently, elimination of tensile stress under prestressing force and dead loads alone is a conservative measure, but one the Committee deemed reasonable and reflective of current practice for prestressed masonry members.

10.4.2 *Allowable stress level load requirements*

10.4.2.1 Because members with laterally unrestrained prestressing tendons are equivalent to members subjected to applied axial loads, the design approach for unreinforced masonry in Section 8.2 has been adopted for convenience and consistency. Buckling of members under prestressing force must be avoided for members with laterally unrestrained prestressing tendons. The prestressing force, P_{ps}, is to be added to the design axial load, P, for stress and load calculations.

10.4.2.2 Lateral restraint of a prestressing tendon is typically provided by grouting the cell or void containing the tendon before or after transfer of prestressing force to the masonry. Lateral restraint may also be provided by placing the masonry in contact with the tendon or the tendon's protective sheath at periodic intervals along the length of the prestressing tendon (Stierwalt and Hamilton (2000)). In general, three intermediate contacts within a laterally unsupported member length or height can be considered to provide full lateral support of the tendon but the analysis and design are the responsibility of the designer.

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10.4.2.3 The prestressing force, P_{ps}, shall be considered for the calculation of the eccentricity of the axial resultant load, *e*, in Equation 8-14.

10.4.3 *Strength requirements*

10.4.3.1 Required strength shall be determined in accordance with the strength ~~level~~ load as designated in Section 4.1.2, except as noted in this~~combinations of the legally adopted building code. When the legally adopted building code does not provide strength level load combinations, structures and members shall be designed to resist the combination of loads specified in ASCE/SEI 7 for strength design~~. Members subject to compressive axial load shall be designed for the strength level design moment and the accompanying strength level axial load. The strength level moment, M_u, shall include the moment induced by relative lateral displacement.

10.4.3.2 The design moment strength shall be taken as the nominal moment strength, M_n, multiplied by a strength-reduction factor (ϕ) of 0.8. The strength-reduction factor (ϕ) for axial load shall also be taken as 0.8. The value of ϕ shall also be taken as 0.8 for the nominal axial load capacity, P_n.

10.4.3.3 *Calculation of f_{ps} for out-of-plane bending of walls*

10.4.3.3.1 For walls loaded out-of-plane and with bonded prestressing tendons, f_{ps} shall be calculated based on strain compatibility, but shall not be larger than f_{py}. In lieu of a more accurate determination, f_{ps} shall be f_{se}.

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Members with laterally restrained prestressing tendons require a modified design approach from the criteria in Section 8.2. If the prestressing tendon is laterally restrained and concentric within the members, the member cannot buckle under its own prestressing force. Any tendency to buckle under prestressing force induces a lateral deformation that is resisted by an equal and opposite restraining force provided by the prestressing tendon. Such members are susceptible to buckling under axial loads other than prestressing, however, and this loading condition must be checked (Scolforo and Borchelt, 1992).

10.4.2.3 For all prestressed masonry members, regardless of tendon restraint and tendon placement, the flexural stress induced by eccentric prestressing causes an increase or decrease in the axial buckling load. Therefore, the prestressing force, P_{ps}, must be considered in the calculation of the eccentricity of the axial resultant, *e*.

10.4.3 *Strength requirements*

Calculation of the moment strength of prestressed masonry members is similar to the method for prestressed concrete members (ACI 318-19 ~~(2014)~~). For bonded tendons, the simplification of taking the tendon stress at nominal moment strength equal to the yield strength can be more conservative for bars than for strands because the yield strength of a prestressing bar is a smaller percentage of the ultimate strength of the tendon.

10.4.3.2 The same value for the strength-reduction factor that is used for tension-controlled sections in flexure (ϕ = 0.8) is used for axial load in prestressed masonry. Axial loads from prestressing will be sufficiently low to ensure ductile behavior if the a/xt limits in Tables 10.5.3 and 10.6.3 are satisfied~~This is the same procedure that is used for reinforced masonry in Chapter 9, where a single value for the strength-reduction factor, ϕ, is used for flexure and axial load~~.

10.4.3.3 *Calculation of f_{ps} for out-of-plane bending of walls*

The equation for the unbonded prestressing tendon stress, f_{ps}, at the moment strength condition (Equation 10-1) is based on tests of walls, which were loaded out-of-plane. Equation 10-1 is used for calculating unbonded tendon stress at nominal moment strength for members loaded out-of-plane containing either laterally restrained or laterally unrestrained tendons. This equation provides improved estimates of the tendon stresses over previous equations in the Code (Bean et al (2007); Bean and Schultz (2003); Bean Popehn (2007); Bean Popehn and Schultz (2010); Bean Popehn and Schultz (2011)). Equation 10-3 is equivalent to the formula in previous editions of the Code except that iterative solution for f_{ps} is not required. The bracketed term in Equation 10-1 is the increase in tendon

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10.4.3.3.2 For walls with laterally restrained or laterally unrestrained unbonded prestressing tendons, Equation 10-1 shall be permitted to be used instead of a more accurate determination of f_{ps}:

$$f_{ps} = f_{se} + \left[\frac{0.03\left(\frac{E_{ps} d}{\ell_p}\right)\left(1 - 1.56\frac{A_{ps} f_{se} + P_u/\phi}{f'_m bd}\right)}{1 - 0.0468\left(\frac{E_{ps} A_{ps}}{f'_m b \ell_p}\right)} \right]$$

$$f_{ps} = f_{se} + \left[\frac{0.03\left(\frac{E_{ps} d_{ps}}{\ell_p}\right)\left(1 - 1.56\frac{A_{ps} f_{se} + P_u/\phi}{f'_m b x_t}\right)}{1 - 0.0468\left(\frac{E_{ps} A_{ps}}{f'_m b \ell_p}\right)} \right]$$

(Equation 10-1)

10.4.3.3.3 In Equation 10-1, the value of f_{ps} shall be not less than f_{se} , and not larger than f_{py}.

10.4.3.4 *Calculation of f_{ps} for walls loaded in-plane (shear walls)* — For shear walls with bonded prestressing tendons, f_{ps} shall be calculated based on strain compatibility, but shall not be larger than f_{py}. Instead of a more accurate determination, f_{ps} for members with unbonded prestressing tendons shall be f_{se} .

10.4.3.5 *Calculation of f_{ps} for beams and lintels* — For beams and lintels with unbonded prestressing tendons, f_{ps} shall be determined based upon Equation 10-1.

10.5 – Design of walls

10.5.1 *Tendons and mild reinforcement centered in wall*

10.5.1.1 For cross sections with uniform width, b, over the depth of the compression zone, the depth of the equivalent compression stress block, a, shall be determined using Equation 10-2:

$$a = \frac{f_{ps} A_{ps} + f_y A_s + P_u/\phi}{0.80 f'_m b} \qquad \text{(Equation 10-2)}$$

For other cross sections, Equation 10-2 shall be modified to consider the variable width of compression zone.

COMMENTARY

stress with flexural deformation of the masonry member as it is loaded to ultimate flexural capacity. The strength level axial load, P_u, in Equation 10-1 reduces the value of f_{ps}, and it is unconservative to neglect this term.

10.4.3.5 *Calculation of f_{ps} for beams and lintels* — García et al. (2017) demonstrated that Equation 10-1 provides conservative estimates for f_{ps} in solid clay masonry beams and ungrouted concrete masonry beams, even though a more accurate formula was developed in this study.

10.5.1.1 The depth of the equivalent compression stress block must be determined with consideration of the cross section of the wall, the tensile resistance of tendons and reinforcement, and the strength level axial load, P_u . P_u is an additive quantity in Equations 10-2 and 10-3. Prestressing adds to the resistance for ultimate strength evaluations and is used with a load factor of 1.0. Equation 10-2 defining the depth of the equivalent compression stress block, a, is modified to match the value for the equivalent uniform stress parameter specified in Chapter 9 (Strength Design of

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10.5.1.2 For walls with uniform width, b, concentric reinforcement and prestressing tendons, and concentric axial load, the nominal moment strength, M_n, shall be calculated using Equation 10-3:

$$\cancel{M_n = \left(f_{ps}A_{ps} + f_y A_s + P_u/\phi\right)\left(d - \frac{a}{2}\right)}$$

$$M_n = \left(f_{ps}A_{ps}\right)\left(d_{ps} - \frac{a}{2}\right) + \left(f_y A_s\right)\left(d - \frac{a}{2}\right) + \left(P_u/\phi\right)\left(x_t - \frac{a}{2}\right) \qquad \text{(Equation 10-3)}$$

10.5.1.3 The quantity a shall be calculated according to Section 10.5.1.1 and f_{ps} shall be calculated according to either Section 10.4.3.4 or Section 10.4.3.5 as applicable.

10.5.1.4 The nominal moment strength for other conditions shall be based on static moment equilibrium principles.

10.5.1.5 ~~The distance d shall be calculated as the actual distance from the centerline of the tendon to the compression face of the member.~~ For walls with laterally unrestrained prestressing tendons and loaded out of plane, ~~d~~ d_{ps} shall not exceed the face-shell thickness plus one-half the tendon diameter plus 0.375 in. (9.5 mm).

10.5.2 *Tendons or mild reinforcement not centered*
When tendons or reinforcement are not placed in the center of the wall, d and d_{ps} shall be calculated in each direction for out-of-plane bending.

10.5.3 The ratio ~~a/d~~ a/x_t shall not exceed the value in Table 10.5.3.

Table 10.5.3: Limits for ~~a/d~~ a/x_t in Prestressed Masonry Walls

Type of Wall	Masonry Unit Material	
	Concrete	Clay
Walls subject to out-of-plane loading, ordinary shear walls	0.36	0.38
Intermediate shear walls	0.23	0.29

COMMENTARY

Masonry) (0.80 f'_m). A review of existing tests of walls indicates that the flexural strength of the walls is more accurately calculated using uniform stresses smaller than the value specified in previous editions of this Code (0.85 f'_m) (Schultz et al (2003); Bean and Schultz (2003)).

10.5.1.2 The equation for the nominal moment strength, M_n, is for the general case of a wall with concentrically applied axial load and concentric tendons and reinforcement. This is representative of most prestressed masonry applications to date. For other conditions, the designer should refer to first principles of structural mechanics to determine the nominal moment strength of the wall.

10.5.2 *Tendons or mild reinforcement not centered*
When tendons or reinforcement are not placed in the center of the wall, d shall be calculated in each direction for out-of-plane bending.

10.5.3 The ratio, a/x_t ~~a/d~~, is limited to assure ductile performance in flexure when using tendons fabricated from steel with yield strengths between 60 ksi (420 MPa) and 270 ksi (1865 MPa). As with reinforced masonry designed in accordance with Chapters 8 and 9, the calculated depth in compression should be compared to the depth available to resist compressive stresses. For sections with uniform width, the value of the compression block depth, a, should be compared to the solid bearing depth available to resist compressive stresses. For hollow sections that are ungrouted or partially grouted, the available depth may be limited to the face shell thickness of the masonry units, particularly if the webs are not mortared. The a/x_t ~~a/d~~ limitation is intended to ensure significant yielding of the prestressing tendons prior to masonry compression failure.

In such a situation, the nominal moment strength is determined by the strength of the prestressing tendon, which is the basis for a strength-reduction factor equal to

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Special shear walls	0.19	0.24

10.6 — Design of beams and lintels

10.6.1 *Compression stress block*
The equivalent compression stress block, a, shall be determined by Equation 10-2.

10.6.2 *Nominal strength*

10.6.2.1 For out-of-plane loading, the centroid of the tension force from the compression face, x_t, shall be computed by Equation 10-4.

$$x_t = \frac{f_{ps} A_{ps}\, d_{ps} + f_y A_s d + P_u t / 2\phi}{f_{ps} A_{ps} + f_y A_s + P_u / \phi} \qquad \text{(Equation 10-4)}$$

10.6.2.2 For in-plane loading, the centroid of the tension force from the compression face, x_t, shall be computed by Equation 10-5.

$$x_t = \frac{f_{ps} A_{ps}\, d_{ps} + f_y A_s d + P_u \ell_w / 2\phi}{f_{ps} A_{ps} + f_y A_s + P_u / \phi} \qquad \text{(Equation 10-5)}$$

10.6.2.3 The nominal moment strength, M_n, shall be calculated by Equation 10-6.

$$M_n = \left(f_{ps} A_{ps} + f_y A_s + \frac{P_u}{\varphi} \right) \left(x_t - \frac{a}{2} \right) \qquad \text{(Equation 10-6)}$$

COMMENTARY

0.8. In previous editions of TMS 402, this limit was determined for sections with bonded tendons.

For masonry with unbonded tendons, the limit on a/x_t ~~a/d~~ was defined using the compressive strain distribution described in Section 9.3.2(g) for strength design of reinforced masonry members, and a tensile strain of $1.5\varepsilon_y$ for walls subjected to out-of-plane loading and ordinary shear walls, a tensile strain of $3\varepsilon_y$ for intermediate shear walls, and a tensile strain of $4\varepsilon_y$ for special shear walls. The only exception is for clay masonry walls subject to out-of-plane loading, and ordinary clay masonry shear walls for which the more conservative limit from previous editions of TMS 402 (i.e., a/x_t ~~a/d~~ $\leq$ 0.38) is used. Because reinforcement strain limits can be applied directly only in the case of bonded tendons, the curvature that is implicit in the strain distribution is used as an indication of flexural deformation. In that manner, a/x_t ~~a/d~~ limits defined using the same strain distribution are imposed on members with either bonded or unbounded tendons. The a/x_t ~~a/d~~ limits in Table 10.5.3 are equal to or more conservative than the value used in previous editions of TMS 402. Thus, Table 10.5.3 was adopted for both bonded tendons and unbonded tendons. Table 10.5.3 is based upon the tendons being in a single layer. There is one ~~d distance~~value for effective depth for walls loaded out-of-plane, and for beams and lintels. However, each tendon has its own ~~d distance~~ effective depth for walls loaded in-plane (shear walls), and the largest value ~~of d~~ should be used for checking the a/x_t ~~a/d~~ limit.

10.6 — Design of beams and lintels

Research on pre-tensioned brick beams conducted in the United Kingdom in the 1970s and 1980s was used to develop the design provisions included in the British masonry code (Schultz and Scolforo 1991). Satisfactory experimental performance to validate the horizontal post-tensioning concept are reported for concrete block beams by García et al. (2019) and for clay brick beams by Baqi et al. (1999). These studies, as well as the analytical investigation by Baqi and Bhandari (2007), confirmed the accuracy of the flexural strength provisions in this Chapter.

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10.6.3 *Ratio of ~~a/d~~ a/x_t*

The ratio *a/d* shall not exceed the value in Table 10.6.3.

Table 10.6.3: Limits for ~~a/d~~ a/x_t in Prestressed Masonry Beams and Lintels

Type of Member	Masonry Unit Material	
	Concrete	Clay
Beams and lintels	0.36	0.38

10.7 — Axial tension

Axial tension shall be resisted by reinforcement, prestressing tendons, or both.

10.8 — Shear

10.8.1 For members without bonded mild reinforcement, nominal shear strength, V_n, shall be calculated in accordance with Sections 9.2.6.1. P_u shall include the effective prestress force, $A_{ps}f_{se}$.

10.8.2 For members with bonded mild reinforcement, nominal shear strength, V_n, shall be calculated in accordance with Section 9.3.3.1.2.

10.8.2.1 Nominal masonry shear strength, V_{nm}, shall be calculated in accordance with Section 9.3.3.1.2.1. P_u shall include the effective prestress force, $A_{ps}f_{se}$.

10.8.2.2 Nominal shear strength provided by reinforcement, V_{ns}, shall be calculated in accordance with Section 9.3.3.1.2.2.

10.9 — Deflection

Calculation of member deflection shall include camber, the effects of time-dependent phenomena, and P-delta effects.

COMMENTARY

10.7 — Axial tension

The axial tensile strength of masonry in a member is to be neglected, which is a conservative measure. This requirement is consistent with that of Section 8.3. If axial tension develops, for example due to wind uplift on the roof structure or due to lateral loading, the axial tension must be resisted by reinforcement, tendons, or both.

10.8 — Shear

This section applies to both in-plane and out-of-plane shear.

The shear strength of prestressed members is calculated using the provisions of the Chapter 9. Calculation of shear strength is dictated by the presence or absence of bonded mild reinforcement. While the Committee acknowledges that many prestressed masonry members are reinforced, for members without bonded mild reinforcement, the unreinforced masonry shear provisions of Chapter 9 are used to calculate shear strength. When bonded mild reinforcement is provided, then the reinforced masonry shear provisions of Chapter 9 are used to calculate shear strength.

No shear strength enhancement due to arching action of the masonry is recognized in this Code for prestressed members. The formation of compression struts and tension ties in prestressed masonry is possible, but this phenomenon has not been considered.

10.9 — Deflection

In accordance with Section 4.~~4~~3.2, prestressed masonry member deflection should be calculated based on uncracked section properties. Calculation of member deflection must include the effect of time-dependent phenomena such as creep and shrinkage of masonry and relaxation of prestressing tendons. There are no limits for the out-of-plane deflection because appropriate out-of-plane deflection limits are project-specific. The designer

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10.10 — Prestressing tendon anchorages, couplers, and end blocks

10.10.1 Prestressing tendons in masonry construction shall be anchored by one of the following:

(a) mechanical anchorage devices bearing directly on masonry or placed inside an end block of concrete or fully grouted masonry, or

(b) bond in reinforced concrete end blocks or members.

10.10.2 Anchorages and couplers for prestressing tendons shall develop at least 95 percent of the specified tensile strength of the prestressing tendons when tested in an unbonded condition, without exceeding anticipated set.

10.10.3 Reinforcement shall be provided in masonry members near anchorages if tensile stresses created by bursting, splitting, and spalling forces induced by the prestressing tendon exceed the capacity of the masonry.

10.10.4 *Bearing stresses*

10.10.4.1 In prestressing tendon anchorage zones, local bearing stress on the masonry shall be calculated based on the contact surface between masonry and the mechanical anchorage device or between masonry and the end block.

10.10.4.2 Bearing stresses on masonry due to maximum jacking force of the prestressing tendon shall not exceed 0.50 f'_{mi}

10.11 — Protection of prestressing tendons and accessories

10.11.1 Prestressing tendons, anchorages, couplers, and end fittings in exterior members exposed to earth or weather, or walls exposed to a mean relative humidity exceeding 75 percent, shall be corrosion-protected.

10.11.2 Corrosion protection of prestressing tendons shall not rely solely on masonry cover.

10.11.3 Parts of prestressing tendons not embedded in masonry shall be provided with mechanical and fire protection equivalent to that of the embedded parts of the tendon.

10.12 — Development of bonded tendons

Development of bonded prestressing tendons in grouted corrugated ducts, anchored in accordance with Section 10.10.1, does not need to be calculated.

COMMENTARY

should consider the potential for damage to interior finishes, and should limit deflections accordingly.

10.10 — Prestressing tendon anchorages, couplers, and end blocks

The provisions of this section of the Code are used to design the tendon anchorages, couplers, and end blocks to withstand the prestressing operation and effectively transfer prestress force to the member without distress to the masonry or the prestressing accessories. Anchorages are designed for adequate pull-out strength from their foundations or other anchorage zones.

Because the actual stresses are quite complicated around post-tensioning anchorages, experimental data, or a refined analysis should be used whenever possible. Appropriate formulas from the references (PTI (2006)) should be used as a guide to size prestressing tendon anchorages when experimental data or more refined analysis are not available. Additional guidance on design and details for post-tensioning anchorage zones is given in Section 25.9 of ACI 318-19 ~~(2014)~~, as well as other references (Sanders et al (1987)).

In most cases, f'_{mi} is equal to or greater than 0.75 f'_m for prestressed masonry. At 0.75 f'_m, the prestressed bearing stress of 0.50 f'_{mi} is equivalent to 0.375 f'_m. If f'_{mi} is specified as equal to f'_m, the maximum permitted bearing stress would be the equivalent of 0.50 f'_m.

10.11 — Protection of prestressing tendons and accessories

Corrosion protection of the prestressing tendon and accessories is required in masonry members subject to a moist and corrosive environment. Methods of corrosion protection are addressed in TMS 602. Masonry and grout cover is not considered adequate protection due to variable permeability and the sensitivity of prestressing tendons to corrosion. The methods of corrosion protection given in TMS 602 provide a minimum level of corrosion protection. The designer may wish to impose more substantial corrosion protection requirements, especially in highly corrosive environments.

10.12 — Development of bonded tendons

Consistent with design practice in prestressed concrete, development of post-tensioned tendons away from the anchorage does not need to be calculated.

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

TMS 402 CODE

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~~6.1, 5.~~5~~6.2(d) and 5.~~3~~4.2.

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 as designated in Section 4.1.2, except as noted in this~~of the legally adopted building code~~. Members subject to compressive axial load shall be designed for the strength level moment accompanying the strength level axial load. The strength level 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~~11.1.8.2. When AAC masonry units with a maximum height of 8 in. (203 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.

COMMENTARY

11.1 — General

11.1.1 *Scope*

Design procedures in Chapter 11 are strength design methods in which internal forces resulting from application of strength level loads must not exceed design strength (nominal member strength reduced by a strength-reduction factor ϕ).

Refer to Section 11.1.10 for requirements for corbels constructed of AAC masonry.

11.1.4 *Strength of joints*

Design provisions of Chapter 11 and prescriptive seismic reinforcement requirements of Chapter 7 are based on monolithic behavior of AAC masonry. The reduction in shear strength of AAC masonry shear walls laid in running bond with unfilled head joints is accounted for in Equation 11-11b. AAC masonry walls constructed with AAC masonry units greater in height than 8 in. (203 mm) (nominal) with unfilled head joints and AAC masonry walls not laid in running bond with unfilled head joints do not have sufficient test data to develop design provisions and thus are not permitted at this time.

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11.1.5 *Strength-reduction factors*

Strength-reduction factors, ϕ, for the design of AAC masonry shall comply with Table 11.1.5.

11.1.6 *Deformation requirements*

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

COMMENTARY

11.1.5 *Strength-reduction factors*

The strength-reduction factor incorporates the difference between the nominal strength provided in accordance with the provisions of Chapter 11 and the expected strength of the as-built AAC masonry. The strength-reduction factor also accounts for the uncertainties in construction, material properties, calculated versus actual member strengths, and anticipated mode of failure.

Anchor bolts embedded in grout in AAC masonry behave like those addressed in Chapter 9 and are designed identically. Anchors for use in AAC masonry units are available from a variety of manufacturers, and nominal resistance should be based on tested capacities.

The value of the strength-reduction factor used in bearing assumes that some degradation has occurred within the masonry material.

The same strength-reduction factor is used for the axial load and the flexural tension or compression induced by bending moment in unreinforced AAC masonry. The lower strength-reduction factor associated with unreinforced AAC masonry (in comparison to reinforced AAC masonry) reflects an increase in the coefficient of variation of the measured strengths of unreinforced AAC masonry when compared to similarly configured reinforced AAC masonry.

The same strength-reduction factor is used for the axial load and the flexural tension or compression induced by bending moment in reinforced AAC masonry. The higher strength-reduction factor associated with reinforced AAC masonry (in comparison to unreinforced AAC masonry) reflects a decrease in the coefficient of variation of the measured strengths of reinforced AAC masonry when compared to similarly configured unreinforced AAC masonry.

Strength-reduction factors for calculating the design shear strength are commonly more conservative than those associated with the design flexural strength. However, the capacity design provisions of Chapter 11 require that shear capacity significantly exceed flexural capacity. Hence, the strength-reduction factor for shear is taken as 0.80, a value 33 percent larger than the historical value.

11.1.6 *Deformation requirements*

11.1.6.1 *Deflection of unreinforced AAC masonry* — The deflection calculations of unreinforced masonry are based on elastic performance of the masonry assemblage as outlined in the design criteria of Section 9.2.2.

Table 11.1.5: Strength-reduction factors for the design of AAC

Condition	Case	Strength-reduction factor, ϕ
Anchor Bolt	Tensile strength of anchor steel	0.75
	Shear strength of anchor steel	0.65
	Anchor bolt pullout	0.65
	AAC masonry breakout, masonry crushing, or anchor pryout	0.50
Bearing	All	0.60
Flexure, Axial Load, or Combinations thereof	Unreinforced masonry	0.60
	Reinforced masonry	0.90
Shear	All	0.80

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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 uncracked 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 290 psi (2.0 MPa).~~

~~**11.1.8.1.2** *Grout compressive strength* — The specified compressive strength of grout, f'_g, shall equal or exceed 2,000 psi (13.8 MPa) and shall not exceed 5,000 psi (34.5 MPa).~~

COMMENTARY

11.1.6.2 *Deflection of reinforced AAC masonry* — Values of I_{eff} are typically about one-half of the uncracked section for common configurations of members. Calculating a more accurate effective moment of inertia using a moment curvature analysis may be desirable for some circumstances. Historically, an effective moment of inertia has been calculated using net cross-sectional area properties and the ratio of the cracking moment strength based on appropriate modulus of rupture values to the applied moment as shown in the following equation. This equation has successfully been used for estimating the post-cracking flexural stiffness of both concrete and masonry.

$$I_{eff} = I_n\left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] \le I_n \le 0.5 I_g$$

11.1.7 *Anchor bolts*

Headed and bent-bar anchor bolts embedded in grout in AAC masonry behave like those addressed in Chapter 9 and are designed identically. Anchors for use in AAC masonry units are available from a variety of manufacturers.

11.1.8 *Material properties*

~~**11.1.8.1** *Compressive strength*~~

~~**11.1.8.1.1** *Masonry compressive strength* — Research (Varela et al (2006); Tanner et al (2005a)), Tanner et al (2005b); Argudo (2003)) has been conducted on structural components of AAC masonry with a compressive strength of 290 to 1,500 psi (2.0 to 10.3 MPa). Design criteria are based on these research results.~~

~~**11.1.8.1.2** *Grout compressive strength* — Because most empirically derived design equations relate the calculated nominal strength as a function of the specified compressive strength of the masonry, the specified~~

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~~11.1.8.2~~11.1.8.1 *Masonry splitting tensile strength* — The splitting tensile strength f_{tAAC} shall be determined by Equation 11-1.

$$f_{t\,AAC} = 2.4\sqrt{f'_{AAC}} \qquad \text{(Equation 11-1)}$$

~~1.1.8.3~~11.1.8.2 *Masonry modulus of rupture* — The modulus of rupture, f_{rAAC}, for AAC masonry 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 50 psi (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 80 psi (552 kPa) at that section.

~~11.1.8.4~~11.1.8.3 *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 50 psi (345 kPa) across an interface between grout and AAC material.

$$\cancel{f_v = 0.15 f'_{AAC}}\; f_{vd} = 0.15 f'_{AAC} \qquad \text{(Equation 11-2)}$$

~~11.1.8.5~~11.1.8.4 *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~~11.1.8.5 *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 60,000 psi (413.7 MPa). The actual yield strength shall not exceed 1.3 multiplied by the specified yield strength.

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~~compressive strength of the grout is required to be at least equal to the specified compressive strength. Additionally, due to the hydrophilic nature of AAC masonry, care should be taken to control grout shrinkage by pre-wetting cells to be grouted or by using other means, such as non-shrink admixtures. Bond between grout and AAC units is equivalent to bond between grout and other masonry units (Tanner et al (2005a), Tanner et al (2005b); Argudo (2003)).~~

~~11.1.8.2~~11.1.8.1 *Masonry splitting tensile strength* — The equation for splitting tensile strength is based on ASTM C1006 tests (Tanner et al (2005b); Argudo (2003)).

~~11.1.8.3~~11.1.8.2 *Masonry modulus of rupture* — The modulus of rupture is based on tests conducted in accordance with ASTM C78 (~~2002~~2021) on AAC masonry with different compressive strengths (Tanner et al (2005b); Argudo (2003); Fouad (2002)). Modulus of rupture tests show that a thin-bed mortar joint can fail before the AAC material indicating that the tensile-bond strength of the thin-bed mortar is less than the modulus of rupture of the AAC. This critical value is 80 psi (552 kPa). The data are consistent with the formation of cracks in thin-bed mortar joints observed in AAC shear wall tests (Tanner et al (2005b); Argudo (2003)). Shear wall tests (Tanner et al 2005b)) show that when a leveling bed is present, flexural cracking capacity may be controlled by the tensile bond strength across the interface between the AAC and the leveling mortar, which is usually less than the modulus of rupture of the AAC material itself.

~~11.1.8.4~~11.1.8.3 *Masonry direct shear strength* — The equation for direct shear strength is based on shear tests (Tanner et al (2005b); Argudo (2003)). Based on tests by Kingsley et al (1985), interface shear strength between grout and conventional masonry units varies from 100 to 250 psi (689 to 1,723 kPa). Based on tests by Forero and Klingner (2011), interface shear strength between grout and AAC material had a 5% fractile (lower characteristic) value of 50 psi (345 kPa).

~~11.1.8.5~~11.1.8.4 *Coefficient of friction* — The coefficient of friction between AAC and AAC is based on direct shear tests performed at The University of Texas at Austin and. the coefficient of friction between AAC and leveling mortar is based on tests on shear walls at the same institution.

~~11.1.8.6~~11.1.8.5 *Reinforcement strength* — Research[3.11] conducted on reinforced masonry components used Grade 60 steel. To be consistent with laboratory documented investigations, design is based on a nominal steel yield strength of 60,000 psi (413.7 MPa). The limitation on the steel yield strength of 130 percent of the nominal yield strength limits the over-strength that may be present in the construction.

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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 4.4~~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:

For AAC floor panels	2 in. (51 mm)
For solid or hollow-core slabs	2 in. (51 mm)
For beams or stemmed members	3 in. (76 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~~6.2(a) through 5.~~5~~6.2(c). The back section of the corbelled section shall remain within ¼ in. (6.4 mm) of plane.

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11.1.9 *Nominal bearing strength*

11.1.9.1 Commentary Section 4.4~~3~~.4 gives further information.

11.1.9.2 *Bearing for simply supported precast floor and roof members on AAC shear walls* — Bearing should be checked wherever floor or roof members rest on AAC walls. The critical edge distance for bearing and the critical section for shear to be used in this calculation are shown in Figure CC-11.1-1.

11.1.10 *Corbels* — Load-bearing corbels of AAC masonry are not permitted due to the possibility of a brittle shear failure. Non-load-bearing corbels of AAC masonry are permitted, provided that the back section of the corbelled wall remains plane within the Code limits. The relative ease in which AAC masonry can be cut and shaped makes this requirement practical.

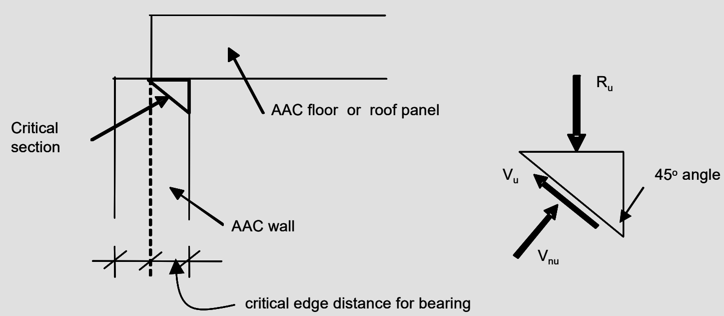

Figure CC-11.1-1 — Critical section at bearing of AAC floor or roof panel on AAC wall

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11.2 — Unreinforced 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 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 AAC masonry members shall be designed to remain uncracked.

11.2.2 *Flexural strength of unreinforced AAC masonry members*

The following assumptions shall apply when determining the flexural strength of unreinforced AAC masonry members:

(a) Design of members for strength level 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 0.85 f'_{AAC}.

(e) The nominal flexural tensile strength of AAC masonry shall be determined from Section ~~11.1.8.3~~11.1.8.2.

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11.2.3 *Nominal axial strength of unreinforced 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:

$$P_n = 0.80\left\{0.85 A_n f'_{AAC}\left[1-\left(\frac{h}{140r}\right)^2\right]\right\} \quad \text{(Equation 11-3)}$$

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

$$P_n = 0.80\left[0.85 A_n f'_{AAC}\left(\frac{70r}{h}\right)^2\right] \quad \text{(Equation 11-4)}$$

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 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.

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11.2.4 *Axial tension*

Commentary Section 8.2.5 provides further information.

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11.3 — Reinforced AAC masonry

11.3.1 *Scope*

This section provides requirements for the design of structures in which reinforcement is used in conjunction with the AAC masonry to resist forces. Design of reinforced AAC masonry by the strength design method shall comply with the requirements of Part 1, Part 2, Section 11.1, and Section 11.3.

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 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~~4.1.4.

(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 $0.85 f'_{AAC}$ 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 $a = 0.67\,c$ 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.

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11.3 — Reinforced AAC masonry

Provisions are identical to those of concrete or clay masonry, with a few exceptions. Only those exceptions are addressed in this Commentary.

11.3.2 *Design assumptions*

For AAC, test results indicate that ε_{mu} for Class 4 AAC masonry and higher is 0.003 and the value of the stress in the equivalent rectangular stress block is $0.85 f'_{AAC}$ with $a = 0.67c$ (Argudo (2003); Tanner et al (2005a)). Additional testing has indicated a ε_{mu} of 0.0012 for Class 2 AAC masonry (Cancino (2003); Tanner et al (2011)).

AAC unit strength class is established using the procedures in ASTM C1693 (2017).

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11.3.3 *Maximum area of flexural tensile reinforcement*

For AAC masonry members where $M_u/(V_u d_v) \geq 1$ of $R \geq 1.5$, 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 AAC masonry as given by Section 11.3.2(c).

(b) The design assumptions of Section 11.3.2 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.

11.3.4 *Design of beams 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 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 11-5 or 11-6. The nominal flexural strength at any section along a member

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11.3.3 *Maximum area of flexural tensile reinforcement* — Longitudinal reinforcement in flexural members is limited to a maximum amount to ensure that AAC masonry compressive strains will not exceed ultimate values. In other words, the compressive zone of the member will not crush before the tensile reinforcement develops the inelastic strain consistent with the curvature ductility implied by the R value used in design.

The maximum reinforcement is determined by considering the prescribed strain distribution, determining the corresponding stress and force distribution, and using statics to sum axial forces.

Because axial force is implicitly considered in the determination of maximum longitudinal reinforcement, inelastic curvature capacity can be relied on no matter the level of axial compressive force. Thus, the strength-reduction factors, ϕ, for axial load and flexure can be the same as for flexure alone. Also, confinement reinforcement is not required because the maximum masonry compressive strain will be less than ultimate values.

The axial force is the expected load at the time of the design earthquake. It is derived from ASCE/SEI 7 Allowable Stress Load Combination 9 and consideration of the horizontal component of the seismic loading. The vertical component of the earthquake load, E_v, should not be included in calculating the axial force for purposes of determining maximum area of flexural tensile reinforcement.

Commentary Section 9.3.5.6.1 provides formulas for determining maximum reinforcement ratios for shear walls subjected to in-plane loads with uniformly distributed reinforcement, for members with only concentrated tension reinforcement, and members with concentrated compression reinforcement that have an area equal to the concentrated tension reinforcement.

For AAC masonry members where $M_u/(V_u d_v) < 1$ and when designed using $R \leq 1.5$, there is no upper limit to the maximum flexural tensile reinforcement.

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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 11-5 or Equation 11-6, as appropriate.

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

$$P_n = 0.80\left[0.85 f'_{AAC}\left(A_n - A_{st}\right) + f_y A_{st}\right]\left[1 - \left(\frac{h}{140r}\right)^2\right]$$

(Equation 11-5)

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

$$P_n = 0.80\left[0.85 f'_{AAC}\left(A_n - A_{st}\right) + f_y A_{st}\right]\left(\frac{70r}{h}\right)^2$$

(Equation 11-6)

11.3.4.1.2 *Nominal shear strength* — Nominal shear strength, V_n, shall be calculated using Equation 11-7 through Equation 11-10, as appropriate.

$$V_n = V_{nAAC} + V_{ns} \quad \text{(Equation 11-7)}$$

where V_n shall not exceed the following:

(a) $V_n = \mu_{AAC} P_u$ (Equation 11-8)

At an interface of AAC and thin-bed mortar or leveling-bed mortar, the nominal sliding shear strength shall be calculated using Equation 11-8 and using the coefficient of friction from Section ~~11.1.8.5~~11.1.8.4.

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

$$V_n \le 6 A_{nv} \sqrt{f'_{AAC}} \quad \text{(Equation 11-9)}$$

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

$$V_n \le 4 A_{nv} \sqrt{f'_{AAC}} \quad \text{(Equation 11-10)}$$

(d) 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 11-11a for AAC masonry with mortared head joints, and Equation 11-11b for masonry with unmortared head joints:

$$V_{nAAC} = 0.95\, l_w\, t \sqrt{f'_{AAC}} \sqrt{1 + \frac{P_u}{2.4\sqrt{f'_{AAC}}\, \ell_w\, t}} \quad \text{and}$$

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11.3.4.1.2 *Nominal shear strength* — The nominal shear strength of AAC walls is based on testing at UT Austin (Tanner et al (2005b); Argudo (2003)). Test results show that factory-installed, welded-wire reinforcement is developed primarily by bearing of the cross-wires on the AAC material, which normally crushes before the longitudinal wires develop significant stress. Therefore, the additional shear strength provided by the horizontal reinforcement should be neglected. Joint-type reinforcement will probably behave similarly and is not recommended. In contrast, deformed reinforcement placed in grouted bond beams is effective and should be included in calculating V_{ns}.

The upper limit on V_n, defined by Equation 11-8, is based on sliding shear. Flexural cracking can result in an unbonded interface, which typically occurs at a horizontal joint in a shear wall. For this reason, the shear capacity of an AAC bed joint is conservatively limited to the frictional resistance, without considering initial adhesion. The sliding shear capacity should be based on the frictional capacity consistent with the perpendicular force on the compressive stress block, including the compressive force required to equilibrate the tensile force in the flexural reinforcement. Dowel action should not be included.

11.3.4.1.2.1 *Nominal masonry shear strength as governed by web-shear cracking* — Equations 11-11a and 11-11b were developed based on observed web shear cracking in shear walls tested at the University of Texas at Austin (Tanner et al (2005b); Argudo (2003)) and Hebel AG (Vratsanou and Langer (2001)) in Germany. Independent testing has validated these equations (Costa et al (2011); Tanner et al (2011)). During testing at the University of Texas at Austin, flexure-shear cracking of AAC shear walls was observed, as predicted, in 6 shear wall tests (Varela et al (2006); Tanner et al (2005a); Tanner et al (2005b)). The presence of flexure-shear cracks did not reduce the strength or stiffness of tested AAC shear walls.

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$$1+\frac{P_u}{2.4\sqrt{f'_{AAC}}\,\ell_w\,t}\geq 0$$

(Equation 11-11a)

$$V_{nAAC}=0.66\,l_w\,t\,\sqrt{f'_{AAC}}\,\sqrt{1+\frac{P_u}{2.4\sqrt{f'_{AAC}}\,\ell_w\,t}}\quad \text{and}$$

$$1+\frac{P_u}{2.4\sqrt{f'_{AAC}}\,\ell_w\,t}\geq 0$$

(Equation 11-11b)

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 11-11c:

$$V_{nAAC}=0.9\sqrt{f'_{AAC}}\,A_{nv}+0.05P_u\geq 0$$

(Equation 11-11c)

P_u shall be considered positive for net compressive axial loads and negative for net tensile axial loads.

11.3.4.1.2.2 *Nominal shear strength as governed by crushing of diagonal compressive strut* — For walls with $M_u/(V_u\,d_v) < 1.5$, nominal shear strength, V_{nAAC}, as governed by crushing of a diagonal strut, shall be calculated as follows:

$$V_{nAAC}=0.17f'_{AAC}t\frac{h\cdot\ell_w^{\,2}}{h^2+(\frac{3}{4}\ell_w)^2}\qquad\text{(Equation 11-12)}$$

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:

$$V_{ns}=0.5\left(\frac{A_v}{s}\right)f_y d_v\qquad\text{(Equation 11-13)}$$

Nominal shear strength provided by reinforcement, V_{ns}, shall include only deformed reinforcement embedded in grout for AAC shear walls.

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Another AAC shear wall tested by Cancino (2003) performed in a similar manner. The results in both testing efforts indicate the hysteretic behavior was not changed after the formation of flexure-shear cracks. Thus, flexure-shear cracking does not constitute a limit state in AAC masonry and design equations are not provided.

Masonry units not laid in running bond may exhibit discontinuities at head joints. The nominal masonry shear strength calculation for AAC masonry not laid in running bond considers the likelihood of vertical discontinuities at head joints and is based on test results for AAC walls made of vertical panels with open vertical joints between some panels.

11.3.4.1.2.2 *Nominal shear strength as governed by crushing of diagonal compressive strut* — This mechanism limits the shear strength at large levels of axial load. It was based on test results (Tanner et al (2005b)), using a diagonal strut width of $0.25\ell_w$ based on test observations.

11.3.4.1.2.3 *Nominal shear strength provided by shear reinforcement* — Equation 11-13 is based on Equation 9-19. Equation 9-19 was developed based on results of reversed cyclic load tests on masonry wall segments with horizontal reinforcement distributed over their heights. The reason for the 0.5 efficiency factor is the non-uniform distribution of tensile strain in the horizontal reinforcement over the height of the member. The formation of an inclined diagonal compressive strut from one corner of the wall segment to the diagonally opposite corner creates a strain field in which the horizontal shear reinforcement at the top and bottom of the segment may not yield. For that reason, not all of the horizontal shear reinforcement in the wall may be fully effective or efficient in resisting shear forces.

AAC masonry walls differ from concrete masonry walls and clay masonry walls in that horizontal joint reinforcement is not used for horizontal shear reinforcement. For reasons of constructability, AAC walls are traditionally reinforced horizontally with deformed steel reinforcing bars in grout-

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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:

$$V_{nAAC} = 0.8\sqrt{f'_{AAC}}\, bd \qquad \text{(Equation 11-14)}$$

11.3.4.2 *Beams* — Design of beams shall meet the requirements of Section 5.~~2~~3 and the additional requirements of Sections 11.3.4.2.1 through 11.3.4.2.5.

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11.3.4.2.1 The strength level axial compressive load on a beam shall not exceed 0.05 $A_n f'_{AAC}$.

11.3.4.2.2 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~~11.1.8.2.

11.3.4.2.3 *Transverse reinforcement* — Transverse reinforcement shall be provided where V_u exceeds ϕV_{nAAC}. The strength level 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-leg stirrup with a 180-degree hook at each end.

(b) Transverse reinforcement shall be hooked around the longitudinal reinforcement.

(c) The minimum transverse reinforcement area divided by its spacing shall be at least 0.0007 *b*.

(d) The first stirrup 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

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filled bond beams. In addition, the strength of the thin set AAC mortar exceeds the strength of the AAC masonry units, which would suggest that AAC walls will behave in a manner similar to reinforced concrete. Assemblage testing conducted on AAC masonry walls also suggested that horizontal joint reinforcement provided in concrete bond beams could be fully effective in resisting shear. For this reason, earlier additions of this Code presented Equation 11-13 without the 0.5 efficiency factor, mimicking the reinforced concrete design equation for strength provided by shear reinforcement.

Although this appeared reasonable in the original judgment of the Committee, no tests have been performed with AAC masonry walls having deformed horizontal reinforcement in concrete bond beams. Until such testing is performed, the 0.5 efficiency factor is being included in Equation 11-13 to be consistent with design procedures associated with concrete masonry and clay masonry, and to provide a conservative design approach.

11.3.4.2.2 Section 9.3.3.2.2.2 permits reducing the minimum tensile reinforcement requirement of 1.3 multiplied by the nominal cracking moment of the beam, M_{cr} to one-third greater than that required by analysis. Because AAC masonry beams tend to be lightly reinforced, this reduction is not appropriate in AAC masonry design.

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one-half the depth of the beam or 48 in. (1219 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 8 in. (203 mm).

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.

11.3.5.3 *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. The nominal axial compressive strength shall not exceed that determined by Equation 11-5 or Equation 11-6, 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 strength level 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 stress from the strength level axial load at the location of maximum moment satisfies the requirement calculated by Equation 11-15.

$$\left(\frac{P_u}{A_g}\right) \leq 0.20 f'_{AAC} \qquad \text{(Equation 11-15)}$$

When the ratio of effective height to nominal thickness, h/t, exceeds 30, the axial stress from the strength level axial load shall not exceed $0.05 f'_{AAC}$.

The strength level moment and axial load shall be determined at the midheight of the wall and shall be used for design. The strength level moment, M_u, at the midheight of the wall shall be calculated using Equation 11-16.

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11.3.5.5.2 This section only includes design equations based on walls having simple support conditions at the top and bottom of the walls. In actual design and construction, there may be varying support conditions, thus changing the curvature of the wall under lateral loading. Through proper calculation and using the principles of mechanics, the points of inflection can be determined and actual moments and deflection can be calculated under different support conditions. The designer should examine moment and deflection conditions to locate the critical section using the assumptions outlined in Section 11.3.5.

The required moment due to lateral loads, eccentricity of axial load, and lateral deformations is assumed maximum at mid-height of the wall. In certain design conditions, such as large eccentricities acting simultaneously with small lateral loads, the design maximum moment may occur elsewhere. When this occurs, the designer should use the maximum moment at the critical section rather than the moment determined from Equation 11-16.

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$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u \qquad \text{(Equation 11-16)}$$

Where:

$$P_u = P_{uw} + P_{uf} \qquad \text{(Equation 11-17)}$$

The deflection due to strength level loads (δ_u) shall be obtained using Equations 11-18 and 11-19.

a) Where $M_u < M_{cr}$

$$\delta_u = \frac{5 M_u h^2}{48 E_{AAC} I_n} \qquad \text{(Equation 11-18)}$$

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

$$\delta_u = \frac{5 M_{cr} h^2}{48 E_{AAC} I_n} + \frac{5\left(M_u - M_{cr}\right) h^2}{48 E_{AAC} I_{cr}} \qquad \text{(Equation 11-19)}$$

11.3.5.5.3 The strength level moment, M_u, shall be determined either by a second-order analysis, or by a first-order analysis and Equations 11-20 through 11-22.

$$M_u = \psi M_{u,0} \qquad \text{(Equation 11-20)}$$

Where $M_{u,0}$ is the strength level moment from first-order analysis.

$$\psi = \frac{1}{1 - \frac{P_u}{P_e}} \qquad \text{(Equation 11-21)}$$

Where:

$$P_e = \frac{\pi^2 E_{AAC} I_{eff}}{h^2} \qquad \text{(Equation 11-22)}$$

For $M_u < M_{cr}$, I_{eff} shall be taken as 0.75 I_n. For $M_u \ge M_{cr}$, 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 11-23, where f_{rAAC} is given by Section ~~11.1.8.3~~11.1.8.2:

$$M_{cr} = S_n\left(f_{rAAC} + \frac{P}{A_n}\right) \qquad \text{(Equation 11-23)}$$

If the section of AAC masonry contains a horizontal leveling bed, the value of f_{rAAC} shall not exceed 50 psi (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 fully grouted wall or a partially grouted wall with the

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11.3.5.5.3 The moment magnifier provisions in this section were developed to provide an alternative to the traditional P-delta methods of Section 11.3.5.5.2. These provisions also allow other second-order analyses to be used.

The proposed moment magnification equation is very similar to that used for slender wall design for reinforced concrete. Concrete design provisions use a factor of 0.75 in the denominator of the moment magnifier to account for uncertainties in the wall stiffness. This factor is retained for uncracked walls. It is not used for cracked walls. Instead, the cracked moment of inertia is conservatively used for the entire wall height. Trial designs indicated that using this approach matches design using Section 11.3.5.5.2. If a 0.75 factor were included along with using the cracked moment of inertia for the entire height would result in design moments approximately 7% greater than using Section 11.3.5.5.2. The Committee did not see any reason for the additional conservatism.

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neutral axis in the face shell shall be obtained from Equation 11-24 and Equation 11-25.

$$I_{cr} = nA_s(d-c)^2 + \frac{nP_u}{f_y}\left(\frac{t_{sp}}{2}-c\right)^2 + \frac{bc^3}{3} \qquad \text{(Equation 11-24)}$$

$$c = \frac{A_s f_y + P_u}{0.57 f'_{AAC}\, b} \qquad \text{(Equation 11-25)}$$

11.3.5.5.6 The design strength for out-of-plane wall loading shall be in accordance with Equation 11-26.

$$M_u \le \varphi M_n \qquad \text{(Equation 11-26)}$$

The nominal moment shall be calculated using Equations 11-27 and 11-28 if the reinforcing steel is placed in the center of the wall.

$$M_n = \left(A_s f_y + P_u\right)\left(d - \frac{a}{2}\right) \qquad \text{(Equation 11-27)}$$

$$a = \frac{\left(P_u + A_s f_y\right)}{0.85 f'_{AAC}\, b} \qquad \text{(Equation 11-28)}$$

11.3.5.6 *Deflections* — The horizontal midheight deflection, δ_s, under allowable stress level loads shall be limited by the relation:

$$\delta_s \le 0.007h \qquad \text{(Equation 11-29)}$$

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 11-18 or Equation 11-19, as applicable, and replacing M_u with M_s 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 11-22.

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 8 ft (2.44 m).

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(b) The maximum reinforcement ratio shall be determined in accordance with Section 11.3.3.

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 11-30, where f_{rAAC} is given by Section ~~11.1.8.3~~11.1.8.2:

$$V_{cr} = \frac{S_n}{h}\left(f_{rAAC} + \frac{P}{A_n}\right) \qquad \text{(Equation 11-30)}$$

If the section of AAC masonry contains a horizontal leveling bed, the value of f_{rAAC} shall not exceed 50 psi (345 kPa).

11.3.6.6 The maximum reinforcement requirements of Section 11.3.3 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:

$$c \geq \frac{\ell_w}{600\left(\delta_{MCE} / h_w\right)} \quad \cancel{c \geq \frac{\ell_w}{600\left(C_d \delta_{ne} / h_w\right)}}$$

and c is calculated for the P_u given by ASCE/SEI 7 Load Combination 6 ($1.2D + E_v + E_h + L +$ ~~0.2~~$0.15S$) or the corresponding strength design load combination of the legally adopted building code, and the corresponding nominal moment strength, M_n, at the base critical section. The load factor on L in Load Combination 6 is reducible to 0.5, as per exceptions to Section 2.3.6 of ASCE/SEI 7.

(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 $M_u/4V_u$.

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11.3.6.6 While requirements for confined boundary elements have not been developed for AAC shear walls, they have not been developed for conventional masonry shear walls either, and the monolithic nature of AAC shear walls favors possible applications involving boundary elements. Also see Commentary Section 9.3.5.6.

11.3.6.6.1 See Commentary Section 9.3.5.6.2.

11.3.6.6.2 See Commentary Section 9.3.5.6.3.

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11.3.6.6.3 Shear walls not designed to the provisions of Section 11.3.6.6.2 shall have special boundary elements at boundaries and edges around openings in shear walls where the maximum extreme fiber compressive stress, corresponding to forces from strength level loads 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 from strength level loads using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in Section ~~5.1.1.1.3~~5.2.3.3 shall be used.

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11.3.6.6.4 Where special boundary elements are required by Section 11.3.6.6.2 or 11.3.6.6.3, (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 $(c - 0.1\ell_w)$ and $c/2$.

(b) In flanged sections, the special boundary element shall include the effective flange width in compression and shall extend at least 12 in. (305 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 12 in. (305 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.

COMMENTARY

11.3.6.6.3 See Commentary Section 9.3.5.6.4.

11.3.6.6.4 See Commentary Section 9.3.5.6.5.

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CHAPTER 12
DESIGN OF MASONRY INFILLS

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12.1 — General

12.1.1 *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 and 5.6, Section 12.1, and either Section 12.2 or 12.3.

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COMMENTARY

12.1 — General

12.1.1 *Scope*

The provisions of Chapter 12 outline a basic set of design provisions for masonry infills based upon experimental research and anecdotal performance of these masonry assemblies. The provisions address both non-participating infills, which are structurally isolated from the lateral force-resisting system, as well as participating infills, which are used to resist in-plane forces due to wind and earthquake. While masonry infills have been a part of contemporary construction for nearly a century, research investigations into their performance, particularly during seismic events, is still ongoing. A comprehensive review of available research data on the performance of masonry infills is provided by Tucker (2007).

As with masonry systems designed by other chapters of this Code, masonry infills must also be designed per the applicable requirements of Part 1 and Part 2. By reference to Part 1, masonry infills must comply with the prescriptive requirements of Chapter 7 for seismic design and detailing. This includes the prescriptive detailing requirements of Section 7.3.1 for non-participating infills and Section 7.3.2 for participating infills. Properly detailed masonry infills have shown considerable system ductility (Henderson et al (2006)). When participating infills are used to resist in-plane loads as part of a concrete or steel frame structure, a hybrid system is effectively created that may not otherwise be defined in Table 12.2-1 of ASCE/SEI 7 for seismic force-resistance. Until further research is completed, the Committee recommends the smallest R and C_d value for the combination of the frame and masonry infill be used to design the system.

Over time, masonry materials expand and contract due to fluctuations in temperature and moisture content as discussed in Commentary Sections 4.2.3, 4.2.4, and 4.2.5. Volumetric changes in the masonry infill will open and close the gap between the infill and the bounding frame, which can have a significant impact on the strength and performance of the infill assembly. Such volumetric changes must be considered as required by Section 4.1.5.

When Chapter 12 (Design of Masonry Infills) was originally developed, information was not available regarding the performance of infills made of AAC masonry and designed according to the provisions of that Appendix. Information has subsequently become available regarding that performance (Ravichandran (2009); Ravichandran and Klingner (2011a); Ravichandran and Klingner (2011b); Ravichandran and Klingner (2011c)). Infills of AAC masonry can safely be

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12.1.2 *Required strength*

Required strength shall be determined in accordance with the strength design load combinations as designated in Section 4.1.2, except as noted in this Chapter~~of the legally adopted building code. When the legally adopted building code does not provide load combinations, structures and members shall be designed to resist the combination of loads specified in ASCE/SEI 7 for strength design~~.

12.1.3 *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 12.1.4.

12.1.4 *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.

12.1.5 *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.

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designed using the provisions of Chapter 12, and using f'_{AAC} instead of f'_m.

While Ravichandran's investigation illustrated that the provisions of Chapter 12 are accurate for stiffness and give conservative (low) values for strength, the user should be aware that underestimating the strength of AAC masonry infill may in turn underestimate the forces that can be transmitted from the infill to the bounding frame. Ravichandran (2009) suggests that this can be addressed by designing the frame for an upper fractile of the calculated infill capacity.

12.1.4 *Strength-reduction factors*

See Commentary Section 9.1.4 for additional discussion on strength reduction factors applicable to concrete and clay masonry. See Commentary Section 11.1.5 for additional discussion on strength reduction factors applicable to AAC masonry. The strength reduction factor applies only to the design of the masonry infill. The strength reduction factors for the anchorage (Section 9.1.4.1 or 11.1.5.1, as appropriate) and bearing (Section 9.1.4.2 or 11.1.5.2, as appropriate) remain unchanged.

12.1.5 *Limitations*

Structures with partial-height infills have generally performed very poorly during seismic events. Partial-height infills create short columns, which attract additional load due to their increased stiffness. This has led to premature column failure. Concrete columns bounding partial-height infills are particularly vulnerable to shear failure (Chiou et al (1999)).

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12.2 — Non-participating infills

Non-participating infills shall comply with the requirements of Sections 12.2.1 and 12.2.2.

12.2.1 *In-plane isolation joints for non-participating infills*

12.2.1.1 In-plane isolation joints shall be designed between the infill and the sides and top of the bounding frame.

12.2.1.2 In-plane isolation joints shall be specified to be at least 3/8 in. (9.5 mm) wide in the plane of the infill, and shall be sized to accommodate the design displacements of the bounding frame.

12.2.1.3 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.

12.2.2 *Design of non-participating infills for out-of-plane loads*

Connectors supporting non-participating infills against out-of-plane loads shall be designed to meet the requirements of Sections 12.2.2.1 through 12.2.2.4. The infill shall be designed to meet the requirements of Section 12.2.2.5.

12.2.2.1 The connectors shall be attached to the bounding frame.

12.2.2.2 The connectors shall not transfer in-plane forces.

12.2.2.3 The connectors shall be designed to satisfy the requirements of ASCE/SEI 7.

12.2.2.4 The connectors shall be spaced at a maximum of 4 ft (1.22 m) along the supported perimeter of the infill.

12.2.2.5 The infill shall be designed to resist out-of-plane 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.

COMMENTARY

12.2.1 *In-plane isolation joints for non-participating infills*

To preclude the unintentional transfer of in-plane loads from the bounding frame to the non-participating infill, gaps are required between the top and sides of the masonry infill assembly. These gaps must be free of materials that could transfer loads between the infill and bounding frame and must be capable of accommodating frame displacements, including inelastic deformation during seismic events.

12.2.2 *Design of non-participating infills for out-of-plane loads*

Mechanical connection between the infill and bounding frame is required for out-of-plane support of the masonry. Masonry infills can be modeled as spanning vertically, horizontally, or both. Connectors between the infill and the bounding frame must be sized and located to maintain load path continuity.

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12.3 — Participating infills

Participating infills shall comply with the requirements of Sections 12.3.1 through 12.3.6.

12.3.1 *General*
Infills with in-plane isolation joints not meeting the requirements of Section 12.2.1 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.

12.3.1.1 The maximum ratio of the nominal vertical dimension to nominal thickness of participating infills shall not exceed 30.

12.3.1.2 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 12.3.1.2.1 or 12.3.1.2.2.

12.3.1.2.1 Where the specified gap between the bounding beam or slab at the top of the infill is less than 3/8 in. (9.5 mm) or the gap is not sized to accommodate design displacements, the infill shall be designed in accordance with Sections 12.3.4 and 12.3.5, except that the calculated stiffness and strength of the infill shall be multiplied by a factor of 0.5.

12.3.1.2.2 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 12.1.5.

12.3.2 *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.

COMMENTARY

12.3.1 *General*
Flanagan and Bennett (1999a) tested an infilled frame with a 1.0 in. gap between the infill and bounding column. Once the gap was closed, the specimen performed like an infilled frame with no gap.

12.3.1.1 The maximum permitted ratio of height to thickness is based on practical conditions for stability.

12.3.1.2.1 Dawe and Seah (1989a) noted a slight decrease in stiffness and strength when a bond breaker (a polyethylene sheet) was used at the top interface. Riddington (1984) showed an approximate 50% decrease in stiffness but little reduction in peak load with a top gap that was 0.1% of the height of the infill. Dawe and Seah (1989a) showed an approximate 50% reduction in stiffness and a 60% reduction in strength with a top gap that was 0.8% of the height of the infill. A top gap that is in compliance with Section 12.2.1.2 is generally less than 0.5% of the infill height. Thus, a 50% reduction in strength and stiffness seems appropriate.

12.3.1.2.2 In cases where the gap at the top of the infill is sufficiently large so that forces cannot be transferred between the bounding frame or beam and the masonry infill, the infill is considered to be partial infill and not permitted to be considered part of the lateral force-resisting system.

12.3.2 *In-plane connection requirements for participating infills*
The modeling provisions of Chapter 12 for participating infills assume that in-plane loads are resisted by the infill by a diagonal compression strut, which does not rely upon mechanical connectors to transfer in-plane load. While mechanical connections, including the use of reinforcement, are permitted, they must be detailed to preclude load transfer between the infill and bounding frame. This is because mechanical connectors between the infill and bounding frame can cause premature damage along the boundaries of the infill under in-plane loading (Dawe and Seah (1989a)). This damage actually reduces the out-of-plane capacity of the infill, as the ability of the infill to have arching action is reduced.

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12.3.3 *Out-of-plane connection requirements for participating infills*

12.3.3.1 Participating infills shall be supported out-of-plane by connectors attached to the bounding frame.

12.3.3.2 Connectors providing out-of-plane support shall be designed to satisfy the requirements of ASCE/SEI 7.

12.3.3.3 Connectors providing out-of-plane support shall be spaced at a maximum of 4 ft (1.22 m) along the supported perimeter of the infill.

12.3.4 *Design of participating infills for in-plane forces*

12.3.4.1 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 12-1; whose thickness is the specified thickness of the infill; and whose elastic modulus is the elastic modulus of the infill.

$$w_{inf} = \frac{0.3}{\lambda_{strut} \cos \theta_{strut}} \qquad \text{(Equation 12-1)}$$

where

$$\lambda_{strut} = \sqrt[4]{\frac{E_m \, t_{net\,\mathrm{inf}} \, \sin 2\theta_{strut}}{4 \, E_{bc} \, I_{bc} \, h_{\mathrm{inf}}}} \qquad \text{(Equation 12-2a)}$$

for the design of concrete masonry and clay masonry infill; and

$$\lambda_{strut} = \sqrt[4]{\frac{E_{AAC} \, t_{net\,\mathrm{inf}} \, \sin 2\theta_{strut}}{4 \, E_{bc} \, I_{bc} \, h_{\mathrm{inf}}}} \qquad \text{(Equation 12-2b)}$$

for the design of AAC masonry infill.

12.3.4.2 Design forces in equivalent struts, as defined in Section 12.3.4.1, shall be determined from an elastic analysis of a braced frame including such equivalent struts.

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12.3.3 *Out-of-plane connection requirements for participating infills*

Mechanical connectors between the bounding frame and the infill provide out-of-plane support of the masonry, for both participating and non-participating infills. Connectors are required only for the direction of span (i.e., at the top of the infill for infill spanning vertically or at the bounding columns for infill spanning horizontally). Connectors for both participating and non-participating infills are not permitted to transfer in-plane loads from the bounding frame to the infill. Clip angles that contact both faces of the infill but do not mechanically anchor to the infill are an example of a mechanical connector that does not transfer in-plane loads from the bounding frame. For a steel bounding frame, the clip angles could be welded to the bottom of the bounding beam (for vertically spanning infill) or to the columns (for horizontally spanning infill). The clip angles could be welded to embedded plates or face fastened to members of a concrete bounding frame.

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12.3.4.3 $V_{n\ inf}$ 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:

(a) $(6.0 \text{ in.}) t_{net\ inf} f'_m$ (Equation 12-3)

(b) the calculated horizontal component of the force in the equivalent strut at a horizontal racking displacement of 1.0 in. (25 mm)

(c) the nominal shear strength, V_n, from Section 9.2.6.1, calculated along a bed joint.

(d) $(6.0 \text{ in.})\ t_{net\ inf} f'_{AAC}$ (Equation 12-4)

(e) the nominal shear strength, V_{nAAC}, from Section 11.2.5, calculated along a bed joint.

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12.3.4.3 The capacity of the infill material is often referred to as corner crushing, although the failure may occur elsewhere as well. Flanagan and Bennett (1999a) compared six methods for determining the strength of the infill material to experimental results of structural clay tile infills in steel frames. The method given in this Code is the simplest method, and also quite accurate, with a coefficient of variation of the ratio of the measured strength to the predicted strength of the infill of 24%. Flanagan and Bennett (2001) examined the performance of this method for predicting the strength of 58 infill tests reported in the literature. Clay tile, clay brick, and concrete masonry infills in both steel and concrete bounding frames were examined. For the 58 tests considered, the coefficient of variation of the ratio of measured to predicted strength of the infill was 21%.

Flanagan and Bennett (1999a) determined that in-plane displacement is a better indicator of infill performance than in-plane drift (displacement divided by height). This was based on comparing the results of approximately 8-ft high (2.4 m) infill tests to 24-ft (7.3 m) high infill tests on similar material. Thus, a displacement limit rather than a drift limit is given in the Code. As a general rule, the strength of the infill is reached at smaller displacements for stiffer bounding columns. For more flexible bounding columns, the strength of the infill is controlled by the displacement limit of 1.0 in. (25 mm).

Section 12.3.4.3.c is intended to address shear failure along a bed joint. The use of a formula from Section 9.2 is not intended to imply that concrete masonry and clay masonry infills are necessarily unreinforced. Shear resistance along a bed joint is similar for the equations of Section 9.2 and Section 9.3, and the former are more clearly related to failure along a bed joint. The same reasoning applies to Section 12.3.4.3.e for AAC masonry infills.

Previous editions of this Code included dividing the nominal shear strengths from Sections 9.2.6 and 11.2.5 by a factor of 1.5 that resulted in overly conservative shear values. This strength reduction was a carryover from Flanagan and Bennett (1999a) where the shear strengths were based on average test values. As noted in the commentary for Section 9.2.6.1, the values calculated using Section 9.2.6 incorporate a two-thirds reduction of the maximum shear stress, thereby eliminating the need to divide those values by 1.5 as previously required. The nominal shear strength values calculated per Section 11.2.5 are based on five-percent lower fractile statistics of the test results. As such, there is no need to further reduce their values.

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12.3.5 *Design of frame members with participating infills for in-plane loads*

12.3.5.1 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.

12.3.5.2 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.

12.3.5.3 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.

12.3.6 *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 12-5a for concrete masonry and clay masonry and Equation 12-5b for AAC masonry:

$$q_{n\inf} = 105\left(f'_m\right)^{0.75} t_{\inf}^2 \left(\frac{\alpha_{arch}}{\ell_{\inf}^{2.5}} + \frac{\beta_{arch}}{h_{\inf}^{2.5}}\right)$$

(Equation 12-5a)

$$q_{n\inf} = 105\left(f'_{AAC}\right)^{0.75} t_{\inf}^2 \left(\frac{\alpha_{arch}}{\ell_{\inf}^{2.5}} + \frac{\beta_{arch}}{h_{\inf}^{2.5}}\right)$$

(Equation 12-5b)

where:

$$\alpha_{arch} = \frac{1}{h_{\inf}}(E_{bc}\, I_{bc}\, h_{\inf}^2)^{0.25} < 35$$

(Equation 12-6)

$$\beta_{arch} = \frac{1}{\ell_{\inf}}(E_{bb}\, I_{bb}\, \ell_{\inf}^2)^{0.25} < 35$$

(Equation 12-7)

In Equation 12-5, 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

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12.3.6 *Design of participating infills for out-of-plane forces*

It is not appropriate to calculate the out-of-plane flexural capacity of unreinforced masonry infills using values for flexural tensile capacity. The predominant out-of-plane resisting mechanism for masonry infills is arching. Even infills with dry-stacked block have been shown to have significant out-of-plane strength (Dawe and Seah (1989b)).

The out-of-plane resistance of a masonry infill as calculated by Equation 12-5 is based upon an arching model of the infill in the bounding frame and therefore neglects the contribution of any reinforcement that may be present in the infill in determining the out-of-plane flexural strength of participating infills. Masonry infills may require reinforcement, however, to resist out-of-plane flexure between points of connection with the bounding frame, or to meet the prescriptive seismic detailing requirements of Chapter 7.

The thickness used in calculations of out-of-plane flexural resistance is limited because infills with low height-to-thickness ratios are less influenced by membrane compression and more influenced by plate bending.

The out-of-plane flexural capacity of the masonry infill is determined based on the work of Dawe and Seah (1989b). They first developed a computer program based on a modified yield line analysis that included the flexibility of the bounding frame. The program coincided quite well with their experimental results, with an average ratio of observed to predicted capacity of 0.98 and a coefficient of variation of 6%. Dawe and Seah (1989b) then used the program for an extensive parametric study that resulted in the empirical equation given here.

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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.

COMMENTARY

Two other equations are available. The first, proposed by Abrams et al. (1993), is used in ASCE 41 (~~2006~~2017). The second was proposed by Klingner et al. (1997). In Flanagan and Bennett (1999b), each of these three proposed equations is checked against the results of 31 experimental tests from seven different test programs including clay brick infills in concrete frames, clay tile infills in steel frames, clay brick infills in steel frames, and concrete masonry infills in steel frames. Flanagan and Bennett (1999b) determined that Dawe and Seah's (1989b) equation is the best predictor of out-of-plane strength, with an average ratio of observed to predicted strength of 0.92, and a coefficient of variation of 0.28. The coefficient of variation of observed to predicted capacity was 28%. Results are summarized in Figure CC-12.3-1. The experimental tests involved infills with height-to-thickness ratios ranging from 6.8 to 35.3. Some infills had joint reinforcement, but this did not affect the results. Two of the specimens had a top gap. Arching still occurred, but was one-way arching. Equation 12-5 is thus quite robust.

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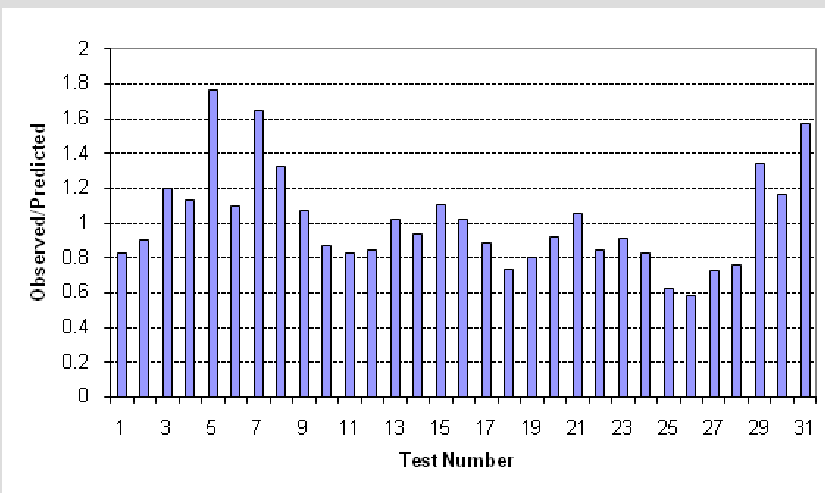

Figure CC-12.3-1 — Ratios of observed to predicted strengths for infills loaded out-of-plane (Flanagan and Bennett 1999b)

PART 4: PRESCRIPTIVE DESIGN METHODS

CHAPTER 13
VENEER

TMS 402 CODE

13.1 — General

13.1.1 *Scope*

This Chapter provides design and detailing requirements for anchored masonry veneer and adhered masonry veneer.

Design of masonry veneer shall comply with the requirements of Part 1, Chapter 4, Section 13.1, and either Section 13.2 or 13.3 except as follows:

(a) TMS 602 Articles 1.4 A and B shall not apply to prescriptively designed anchored and adhered masonry veneer.

(b) TMS 602 Articles 3.4 C, D and F shall not apply to anchored and adhered masonry veneer.

(c) TMS 602 Article 3.4 B, except Article 3.4 B.11, shall not apply to anchored masonry veneer.

(d) Section 4.7~~6~~ and TMS 602 Articles 3.3 B and 3.4 A, B, and ~~E~~ D shall not apply to adhered masonry veneer.

COMMENTARY

13.1 — General

13.1.1 *Scope*

The scope of this Code is masonry consisting of masonry units bedded in mortar (Section 1.1.1). Thus, large stone veneer, slab-type veneer, and other veneer systems not bedded in mortar are not covered by this Chapter. Table CC-13.1.1 lists the permitted design methods for the various masonry materials. More specific requirements on each material and design method can be found within the respective sections. Cast stone not bedded in mortar and not using veneer ties as required by this Code can be designed using TMS 404/504/604.

Veneer is not considered to add strength or stiffness to the wall system. If the masonry wythe and its backing are designed to share load, then it is not considered a masonry veneer wall, and the masonry would need to be designed by other portions of this Code. The design of the backing should be in compliance with the appropriate standard for the backing material.

Because veneer is non-load-bearing, there is no need to specify the compressive strength of masonry veneer, although the specified compressive strength, f'_m, may be used for some aspects of Engineered Design in this chapter.

TMS 602 articles that are excluded address materials and requirements that are not applicable to veneer construction or are items addressed by specific requirements in this Chapter and are put here to be inclusive.

Failures of anchored and adhered veneer are often due to nonconformance with the contract documents. Therefore, TMS 602 Table 4 requires periodic inspection when the height of the veneer exceeds 60 ft (18.3 m) above grade plane. Consideration should be given to inspection for all veneers with the level of inspection varying by the job.

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Commented [PJS2]: 19-VG-064

COMMENTARY

Table CC-13.1.1: Permitted Materials for each Design Method [1]

Masonry Material	Anchored Veneer		Adhered Veneer	
	Prescriptive	Engineered	Prescriptive	Engineered
Clay and Concrete	X	X	X	X
Dimension Stone		X	X	X
Cast Stone	X	X	X	X
Manufactured Stone			X	X

1 Specific requirements for each of these materials can be found in the respective design method sections.

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13.1.2 *General design requirements*

13.1.2.1 Design and detail wall systems to comply with weather, structural, fire, and thermal resistance requirements of the legally adopted building code.

13.1.2.2 *Deformation and differential movement*

COMMENTARY

13.1.2 *General design requirements*

13.1.2.1 Veneer, being part of the building envelope, is subject to many other requirements in the legally adopted building code besides the requirements in this Code. These other requirements may control the design of the veneer.

Detailing adhered masonry veneers for these design considerations can vary from anchored masonry veneers. Numerous suggested details for adhered masonry veneers are in ~~MVMA~~ NCMA (~~2017~~2021) and BIA TN 28C (2014).

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13.1.2.2 *Deformation and differential movement* — Deformations include, but are not limited to, out-of-plane deflection of the backing, vertical deflection of horizontally spanning support elements, and in-plane movement due to absolute and relative story drift. See Sections 13.2.1.5 and 13.3.1.2 for deflection requirements specific to anchored and adhered veneers, respectively.

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Building drift due to wind and seismic can have an impact on veneer design and will vary greatly depending on factors such as type of building frame (and its flexibility), building size, shape and weight. The minimum drift criteria set by building codes account for several factors, including the variability of actual loads and stiffness and the larger drifts that will occur due to inelastic deformations associated with major earthquakes. Significant drift can also occur during wind events and should also be considered. Isolation and panelization of the masonry veneer are strategies to mitigate the effects due to drift, such as force transfer at veneer boundaries or through the veneer ties. Veneer ties which allow horizontal in-plane movement can allow modest movement to occur without loading the veneer. Masonry veneer at building corners are especially vulnerable to damage from story drift, consequently, they should receive extra attention in design.

There are many aspects of differential movement that need to be considered in the design of masonry veneer. One is the movement within the veneer itself due to irreversible moisture expansion, shrinkage, creep, and temperature movements. With anchored veneers, these movements are typically accommodated with expansion joints in clay masonry and control joints in concrete masonry. Information on the design and detailing of expansion joints can be found in BIA TN 18A (2019) and in NCMA TEK 10-4 (2001) for control joints.

Consideration needs to be given to designing for differential movement between different support conditions and between the veneer and the structure at openings and penetrations. This is particularly critical with anchored clay masonry veneer (which expands) with wood light frame backing (which shrinks). See Section 13.1.2.2.2 for further information.

Grimm and Yura (1989) provide information and recommendations for design of shelf angles in anchored veneers, including accommodating frame deformations in the design of the expansion joint.

Masonry veneer can be designed with horizontal and vertical bands of different materials. The dissimilar physical properties of the materials should be considered when deciding how to accommodate differential movement. Industry recommendations are available regarding horizontal bands of clay masonry and concrete masonry, and address such items as ~~joint~~ reinforcement, slip planes, and sealant joints (NCMA TEK 5-2A (2002); BIA TN 18A (2019)). Vertical movement joints can be used to accommodate differential movement between vertical bands of dissimilar materials.

Movement joints should be considered in adhered veneer where differential movement is expected. Examples are at the transition between a light frame backing assembly and a concrete masonry backing assembly, transitions between adhered veneer and other exterior coverings, and at openings. Recommendations for movement joint spacing and detailing for adhered masonry veneers can be found in ~~MVMA~~ NCMA (~~2017~~2021) and BIA TN 28C (2014).

13.1.2.2.1 Veneer shall be designed and detailed to accommodate deformations and differential movement.

13.1.2.2.2 *Wood light frame backing* — Exterior veneer ~~tied~~ connected to wood light frame construction exceeding 30 ft (9.1 m), or 38 ft (11.58 m) at a gable, in height above the vertical support shall be designed and detailed to accommodate differential movement.

13.1.2.2.2 *Wood light frame backing* — When veneer with a backing of wood exceeds 30 ft (9.1 m), or 38 ft (11.58 m) at a gable, in height, design and detailing for differential movement between the wood light frame backing and masonry veneer is critical to the performance of the masonry veneer. Alternative framing, such as balloon framing instead of platform framing, is one option to limit the shrinkage of the wood frame. Detailing around openings and penetrations through the veneer needs to be carefully considered. Information on conducting an analysis for heights exceeding 30 ft (9.1 m) and proper

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detailing are given in Silvester et al (2014) and Clark et al (2015).

13.1.2.3 *Deflection of horizontally spanning support members* — Horizontally spanning members supporting veneer shall be designed so that the vertical deflection due to allowable stress level dead plus live loads does not exceed l/600.

13.1.2.3 *Deflection of horizontally spanning support members* — See Commentary Section ~~4.6~~4.56 for further information.

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13.1.2.4 *Limitation of applied vertical loads other than self-weight* — Superimposed allowable stress level vertical loads on the face of the veneer shall not exceed ~~20 pounds (89 N) vertical load applied~~ the following in any 5 ft (1.52 m) by 5 ft (1.52 m) wall face area.

(a) 20 pounds (89 N) vertical load, and

(b) 180 in.-lb (20,340 N-mm) moment. Items attached to the veneer that do not project more than 12 in. (305 mm) from the face of the veneer shall be deemed to comply with this moment limitation.

~~Items attached to the veneer shall not project more than 12 in. (305 mm) from the face of the veneer.~~ Exception: This load limit may be exceeded for anchored veneers designed in accordance with Section 13.2.3.3 provided that the masonry is designed in accordance with Parts 2 and 3 of this Code.

13.1.2.4 *Limitation of applied vertical loads other than self-weight* — Because veneer is non-structural, it is not designed to carry superimposed loads. This provision allows attaching small items, such as address plates and porch lights, to the veneer. Attachments that project more than 12 in. (305 mm) from the face of the veneer can have significant snow and wind loads and the designer should consider that possibility. If the applied load or attachment projection exceeds these limits, the masonry veneer and veneer ties would need to be designed using ~~either Chapter 8 or Chapter 9~~ Parts 2 and 3 of this Code ~~and form the basis of an Engineered Design~~. In particular, the masonry would have to be designed using the unreinforced masonry provisions (Section 8.2 or Section 9.2) which require the masonry to be designed to remain uncracked, or as reinforced masonry (Section 8.3 or Section 9.3). The other provisions of Chapter 13, such as the stability provisions, would ~~still~~ apply. The additional ~~tie~~ forces from the applied loads also need to be considered. ~~In an Engineered Design, masonry wythe stability should be considered if these limits are exceeded.~~

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These requirements apply only to items attached to the veneer, not items that pass through the veneer and are supported by the backing.

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13.2 — Anchored veneer

13.2.1 *General requirements for anchored veneer*

13.2.1.1 *Scope* — This section provides requirements for the design of anchored veneer as defined in Section 2.2. The Design method utilized shall comply with Table 13.2.1.1 based on wind pressure and Seismic Design Category of the structure. Anchored veneer shall be designed by either:

(a) the Prescriptive Design method of Section 13.2.2, using the basic veneer tie requirements or enhanced veneer tie requirements, or

(b) the Engineered Design method of Section 13.2.3.

COMMENTARY

13.2 — Anchored veneer

13.2.1 *General requirements for anchored veneer*

Anchored veneer transfers out-of-plane loads through the veneer ties to the backing. The backing accepts and resists the veneer tie loads and is designed to resist the out-of-plane loads.

When utilizing anchored masonry veneer, the designer should consider the following conditions and assumptions:

a) Veneer may crack in flexure under allowable stress level loads.

b) Deflection of the backing should be limited to control crack width in the veneer and to provide veneer stability.

c) Connections of the veneer tie to the veneer and to the backing should be sufficient to transfer applied loads.

d) Differential movement should be considered in the design, detailing, and construction.

e) Water will penetrate the veneer, and the wall system should be designed, detailed, and constructed to prevent water penetration ~~into the building~~beyond the drainage space and insulation.

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f) Corrosion and fire resistance should be considered as required by the governing building code.

If the exterior masonry wythe is not considered to add strength to the wall in resisting out-of-plane load, then the exterior wythe is masonry veneer. However, if the exterior wythe is considered to add strength to the wall in resisting out-of-plane load, then the wall is properly termed either a multiwythe, non-composite or composite wall rather than a masonry veneer and its design should conform to other portions of this Code.

13.2.1.1 *Scope* — The anchored veneer requirements consist of two alternatives, each with two options:

(a) Prescriptive Design per Section 13.2.2 using the basic or enhanced veneer tie requirements;

(b) Engineered Design method using either the Tributary Area method per Section 13.2.3.2, which is a relatively simple analytical design method taking into consideration veneer tie stiffness and veneer geometry in the distribution of veneer tie forces and spacing; or Modeling Analysis method per Section 13.2.3.3, applicable for more complex or unique veneer applications, systems, or materials.

The prescriptive design options for anchored veneer were verified by the Committee for both out-of-plane wind and seismic forces, with wind producing the controlling design load. The design of a more economical veneer tie is possible at discrete zones across a building's surface where lower design wind pressures exist. However, in accordance with the requirements of ASCE/SEI 7, corner negative pressures are required to be applied to the entire height of

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the building. For a parapet, the masonry veneer is designed only for the pressure on the surface of the veneer, and not the total force on the windward and leeward side of the parapet.

Where Basic veneer tie spacing requirements are permitted by Table 13.2.1.1, Enhanced veneer tie spacing requirements are also permitted.

13.2.1.2 *Masonry units* — Masonry units shall be at least 2.625 in. (66.7 mm) in specified thickness.

13.2.1.2 *Masonry units* — Given the cover requirements for corrosion protection and required embedment of veneer ties to resist load, this is the minimum practical thickness of a masonry unit that can be used for anchored veneer.

13.2.1.3 *Veneer not laid in running bond* — Anchored veneer not laid in running bond shall be reinforced with veneer wire reinforcement that conforms to one of the following: ~~have joint reinforcement of~~

(a) at least one wire, of size W1.7 (MW11) with deformations knurled in conformance with ASTM A951

(b) at least one deformed wire of minimum size D 2 (MD 13)

(c) joint reinforcement of minimum size W1.7 (MW11)

The reinforcement shall meet the minimum area requirements of Section 4.6 and be~~,~~ spaced at a maximum of 18 in. (457 mm) on center vertically.

13.2.1.3 *Veneer not laid in running bond* — ~~The required area of joint reinforcement is equivalent to that in Section 4.5~~4.6 ~~for a nominal 4 in. (102 mm) wythe.~~For commonly available W1.7 (MW11) wire this would correspond to a maximum spacing of 18 in. (457 mm) for nominal 3 in. (76 mm) wythes and 16 in. (406 mm) for nominal 4 in (102 mm) wythes.

13.2.1.4 *Joint thickness for veneer ties* — ~~Specified mortar bed joint thickness shall be at least twice the thickness of the specified embedded veneer tie.~~

13.2.1.4.1 For specified veneer ties that rely on embedment in mortar for strength, the specified mortar bed joint thickness shall be at least twice the thickness of the veneer tie. If the joint also has veneer wire reinforcement stacked on the veneer tie, the specified mortar bed joint thickness shall be at least twice the combined thickness.

13.2.1.4.2 For veneer ties that utilize a mechanical connector to engage veneer wire reinforcement

13.2.1.4 *Joint thickness for veneer ties* — This provision is not intended to prohibit the placement of veneer wire~~joint~~ reinforcement and veneer ties in the same bed joint~~, but they must not be stacked to exceed the maximum joint thickness~~. Three options exist:

(a) Veneer ties that rely on mortar embedment can be placed side by side with the veneer wire reinforcement such that the minimum joint thickness is at least twice the thickness of either.

(b) Veneer ties that rely on mortar embedment can be stacked but not connected to the veneer wire reinforcement provided the minimum joint thickness is at least twice their combined thickness.

(c) Veneer ties that rely upon mechanical anchorage with the veneer wire reinforcement can be used provided the mortar joint is thicker than the combined thickness of the veneer tie and wire and no less than twice the thickness of the veneer wire reinforcement.

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for anchorage, the specified mortar joint thickness shall be greater than the combined thickness but no less than twice the thickness of the veneer wire reinforcement.

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Table 13.2.1.1: Permitted design methods for anchored veneer

p_{veneer}, psf (kPa) [1]	Permitted Design Method [2]	
	Seismic Design Category A, B, and C	Seismic Design Category D and higher
≤ 50 (2.39)	Prescriptive (Section 13.2.2 Basic) or Engineered (Section 13.2.3)	Prescriptive (Section 13.2.2 Enhanced) or Engineered (Section 13.2.3)
> 50 (2.39) and ≤ 75 (3.59)	Prescriptive (Section 13.2.2 Enhanced) or Engineered (Section 13.2.3)	
> 75 (3.59)	Engineered (Section 13.2.3)	

[1] p_{veneer} is determined from ASCE/SEI 7, Chapter 30.

[2] Section 13.2.2 Basic and Section 13.2.2 Enhanced refer to veneer tie spacing requirements in Table 13.2.2.5.

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13.2.1.5 *Out-of-plane deflection* — For the purpose of maintaining veneer stability, backings shall have a calculated deflection less than or equal to those in Table 13.2.1.5, or a stability analysis shall be performed to demonstrate a minimum factor of safety of 1.5 against loss of stability under strength level loading. The veneer ties and backing shall be designed for any additional forces determined from the stability analysis.

Table 13.2.1.5: Maximum deflection of the backing to provide out-of-plane stability

h_b/t_{sp}	Maximum Deflection of the Backing for Stability	
	Wind[1], δ_{ser}	Seismic[2], δ_u
67	$h_b/240$	$h_b/100$
100	$h_b/360$	$h_b/150$
133	$h_b/480$	$h_b/200$
167	$h_b/600$	$h_b/250$

[1]Under application of 0.42 times the strength level wind load and applicable to backing whose stiffness is the same for service level and strength level wind loads. If the stiffness is not the same, evaluate stability using strength level wind loads and using the deflection limits for seismic loads.

[2]Under application of the strength level seismic load.

COMMENTARY

13.2.1.5 *Out-of-plane deflection* — The deflection of the backing creates the potential for instability in the veneer by creating an eccentricity between the weight of the veneer and the geometric area of the veneer that is available to resist that load. Figure CC-13.2-1 depicts the condition producing the greatest eccentricity of the vertical load on the section at mid-height, which will occur when the veneer is cracked at mid height. Veneers that remain elastic or have multiple levels of cracking will have a reduced eccentricity at mid-height. If the strength level displacement of the veneer at mid-height due to the displacement of the backing, δ_u, is expressed as a ratio of the height of the backing, h_b/x (for example $h_b/360$), the resulting eccentricity of the veneer weight is $h_b/2x$. As shown in Figure CC-13.2-1, the maximum permitted eccentricity of the load before loss of stability is $t_{sp}/2$.

In order to maintain a factor of safety of 1.5 against loss of stability, the maximum eccentricity of the veneer gravity load is limited to $t_{sp}/3$. Equating $t_{sp}/3$ to $h_b/2x$ results in x being equal to $3/2(h_b/t_{sp})$. The value of h_b is the effective height of the backing, and not the distance between supports of the veneer, as shown in Figure CC-13.2-2.

The deflection obtained from $h_b/x=2h_b/(3(h_b/t_{sp}))$ is the deflection at strength level loads and is appropriate for use with seismic loads. With wind loads, the stiffness of the backing is often evaluated at service level wind loads. The service level wind load is taken as 42% of the ultimate wind load in accordance with footnote "f" of IBC (2021) Table 1604.3 and corresponds to a 10-year mean return wind speed. The wind deflection limits in Table 13.2.1.5 correspond to the same strength level deflection as the seismic deflection limits and were determined by multiplying the strength level deflection by 0.42. This is only applicable to backing whose stiffness remains the same for service level and strength level wind loads. Examples of backings whose stiffness would not remain the same are a concrete or masonry backing that cracks, or a backing that has significant second-order effects at strength level loads.

If the backing does not meet the deemed to comply deflection criteria of Table 13.2.1.5, the designer can choose to stiffen the backing, or account for the additional forces in the veneer ties and backing to provide stability.

The deflection limits of Table 13.2.1.5 are only for stability. Deflection limits for serviceability may differ from those for stability and should be established on a project specific basis. There have been various recommendations for maximum serviceability deflection limits, with the recommendations generally in the range of $h_b/600$ to $h_b/360$ (Clark et al, 2018a). These values were developed for metal stud backing but are applicable to all backings. Stiffness values can be approximately related to crack width based on geometry (Clark et al, 2018b).

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$$w_{cr} = \frac{4\delta t_{sp}}{h_b}$$

where w_{cr} is the crack width, δ is the deflection of the backing, t_{sp} is the thickness of the veneer, and h_b is the height of the backing. For a veneer thickness of 3.625 in. (92.1 mm), a backing deflection of $h_b/600$ will result in a crack width of approximately 0.02 in. (0.5 mm) and a backing deflection of $h_b/360$ will result in a crack width of approximately 0.04 in. (1 mm).

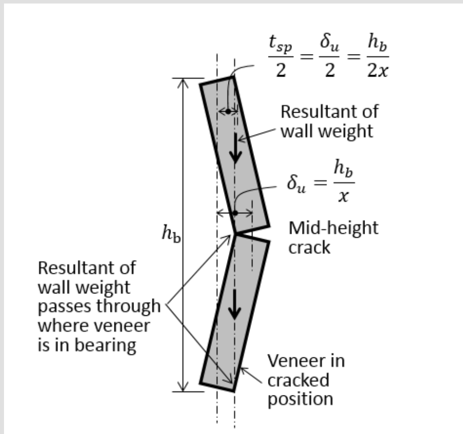

Figure CC-13.2-1 — Determination of limiting h_b/t_{sp}

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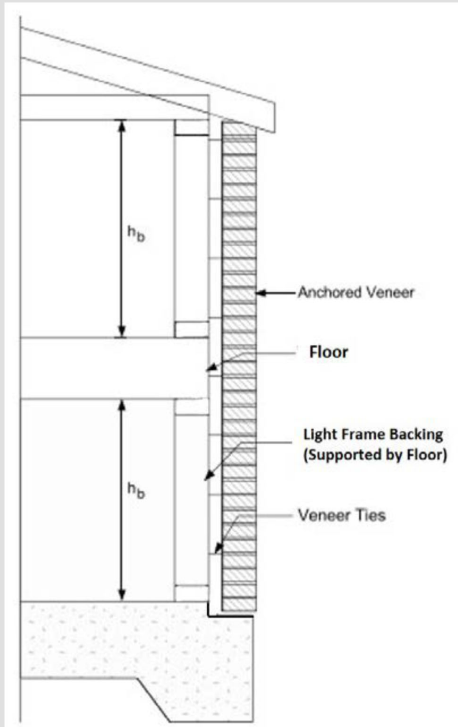

Figure CC-13.2-2 — Determination of height of backing, h_b

13.2.1.6 *Support above openings* — Lintels, shelf angles, or arches shall be designed above openings.

13.2.1.6 *Support above openings* — The design of lintels should be based on the various material codes and standards for that type of support. Minimum bearing lengths will vary with the appropriate codes and standards but will often be at least 4 in. (102 mm) in the direction of span. Masonry lintels can be designed using Chapter 5 and other references (McGinley et al (2003)).

13.2.1.7 *Seismic* — In Seismic Design Categories C, D, E and F, the sides and top of anchored veneer shall be isolated from the structure so that vertical and lateral seismic forces resisted by the structure are not imparted to the veneer. In Seismic Design Categories E and F, the weight of anchored veneer for each story shall be supported independent of other stories.

13.2.1.7 *Seismic* — The isolation from the structure reduces accidental loading and permits larger building deflections to occur without veneer damage. Support at each floor articulates the veneer and reduces the size of potentially damaged areas, particularly in structures with large relative story drifts. Added movement joints further articulate the veneer, permit greater building deflection without veneer damage and limit stress development in the veneer.

13.2.1.8 *Water Penetration Resistance* — Flashing and weep holes in exterior veneer wall systems shall be designed and detailed to resist penetration of water ~~into the building interior~~beyond the drainage space and

13.2.1.8 *Water Penetration Resistance* — Water penetration through the exterior veneer is expected. The wall system must be designed and constructed to prevent

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insulation. A minimum 1 in. (25.4 mm) drainage space shall be specified. Weep holes shall be at least 3/16 in. (4.8 mm) in diameter and spaced less than 33 in. (838 mm) on center.

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water from ~~entering the building~~ passing beyond the drainage space and insulation.

A drainage space, flashing system and weep holes are required to remove moisture from behind the veneer. The minimum drainage requirement is intended to provide drainage of the cavity and to allow the mason to grip and install the masonry veneer unit. Recognizing there will be localized restrictions within the cavity, the designer is cautioned to provide adequate drainage. Restrictions may occur at veneer ties, shelf angles and weep holes or may result from allowable tolerances as shown in Figure CC-13.2-3. Adequate drainage at restrictions can be achieved by increasing the drainage space width beyond the 1 in. (25.4 mm) minimum, adding a drainage device, or using alternative design and construction.

In addition to a drainage space, flashing and weeps, incorporating air movement in a masonry wall to create a rainscreen is a good design strategy. Weeps that permit airflow into the cavity can be used to assist in removing moisture from a veneer wall. Improved performance can be achieved by adding vents at the top of cavity compartments or near the top of the wall to further aid in evaporation and drying (BIA TN 27 (1994)).

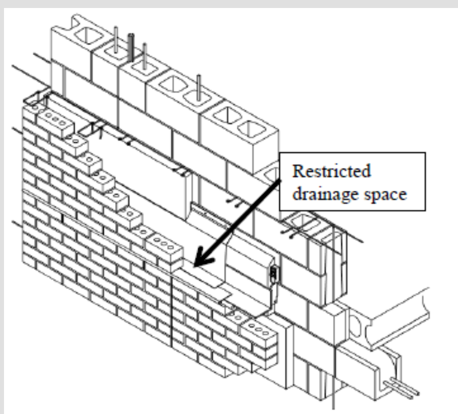

Figure CC-13.2-3 — Example of drainage space restriction

13.2.2 *Prescriptive design of anchored masonry veneer*

13.2.2.1 *Permitted units* — Prescriptively-designed anchored veneer shall be constructed of units complying with:

(a) TMS 602 Article 2.3 A
(b) TMS 602 Article 2.3 B except ASTM C34, ASTM C56 and ASTM C1088
(c) TMS 602 Article 2.3 C
(d) TMS 602 Article 2.3 F

13.2.2 *Prescriptive design of anchored masonry veneer*

13.2.2.1 *Permitted units* — The prescriptive requirements of anchored masonry veneers apply to conventional concrete masonry, clay masonry, dimension stone and cast stone masonry units laid in mortar that also meet the other limitations of Section 13.2.2. Alternative anchored veneer systems would need to be evaluated and designed in accordance with Section 13.2.3.

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13.2.2.2 *Specified weight, height, and thickness* — Prescriptively-designed anchored veneer units shall have a:
(a) specified weight of 50 psf (2.4 kPa) or less,
(b) specified height of 16 in. (406 mm) or less, and
(c) ~~a~~ specified thickness of 5 in. (127 mm) or less.

13.2.2.3 *General requirements* — Prescriptively-designed anchored veneer shall comply with the requirements of Table 13.2.2.3. Prescriptively-designed dimension stone anchored masonry veneer shall not exceed 30 ft (9.1 m) in height above grade plane.

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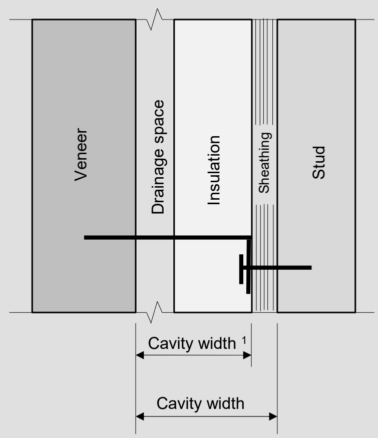
13.2.2.2 *Specified weight, height, and thickness* ~~*Installed weight*~~ — The limitation on installed weight is to limit the force under seismic load, when seismic loading is a concern. Units that have an average thickness greater than 5 in. (127 mm) require an engineered analysis.

13.2.2.3 *General requirements* — The cross-section of a typical anchored veneer wall where the backing is either wood or metal studs is shown in Figure CC-13.2-4. A minimum 1 in. (25.4 mm) drainage space is required per Section 13.2.1.8.



1 When the requirements of Section 13.2.2.3.3 are met.

Figure CC-13.2-4 — Cross-section of typical anchored veneer supported by light ~~frame~~framing

For backings supporting masonry veneer that are not listed in Table 13.2.2.3 the Engineered Design method (Section 13.2.3) should be used.

Veneers higher than 30 ft (9.1 m), or 38 ft (11.58 m) at a gable, are permitted with wood and cold-formed metal light frame backing provided a veneer tie other than corrugated sheet-metal is used, and detailing is provided to account for the differential movement. Support of veneer with a wood or cold-formed ~~steel~~ metal light frame backing typically occurs at grade level; however, it may also occur at the top of a noncombustible podium when podium-type construction is used. For flexible and taller structures, the differential lateral movement of the veneer and supporting structure must be addressed in the design, typically through appropriate detailing of movement joints. The height limitation is measured from the point of support wherever that may occur. For most structures, vertical differential movement is often accommodated by supporting the veneer at each story above 30 ft (9.1 m) with a shelf angle. See Commentary Section 13.1.2.2.2 for further information on brick veneer on wood light frame backing exceeding 30 ft

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(9.1 m) in height since shelf angles may not always be included in wood light frame backing structures.

The maximum cavity width of 1 in. (25.4 mm) for corrugated sheet-metal veneer ties and the minimum 1 in. (25.4 mm) drainage space requirement may be accommodated within the tolerance for each requirement. Other veneer ties may provide more options.

A ring-shank nail designated as Roof Sheathing Ring-Shank (RSRS-03) nail has a length of 2.5 in. (63.5 mm) and a diameter of 0.131 in. (3.33 mm) and is similar to an 8d nail. When more than one nail is required, a screw that provides greater withdrawal resistance should be used or two nails with some qualifications. If two nails or screws are used the designer should consider whether the fasteners can be placed side by side or if the backing plate is stiff enough in a vertical direction to equally load both fasteners. Prescriptive design requires the fastener to have sufficient penetration into the backing and does not consider the sheathing to contribute to pullout resistance.

~~Although Table 13.2.2.3 contains only one deemed-to-comply fastener requirement for masonry and concrete backings, other fasteners with equivalent strength can be used, but would need to be verified with an engineered design.~~

Due to empirical results and industry recommendations, dimension stone veneer height limitations should not exceed the limits stated. Dimension stone masonry walls that would exceed these limits must use the Engineered Methods in Section 13.2.3 or Section 1.3 Alternative design or method of construction.

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Table 13.2.2.3: General prescriptive anchored veneer requirements

Backing	Veneer Tie Type	Maximum Specified Cavity Width	Other requirements
Wood Light ~~Frame~~Framing	Corrugated Sheet-metal	1 in. (25.4 mm) [1]	Fastener: Minimum 2.5 in. (63.5 mm) x 0.131 in. (3.33 mm) ring-shank nail(s) with minimum 1 $^{3}/_{8}$ in. (34.9 mm) penetration into backing or No. 10 screw(s) with $^{5}/_{8}$ in. (15.9 mm) penetration into backing. Where sheathing is present, the minimum penetration shall be into the structural member behind the sheathing. Locate fastener within ½ in. (12.7 mm) of the 90-degree bend in the veneer tie. The limiting p_{veneer} values for prescriptive design method shall be 75 percent of those listed in Table 13.2.1.1. Corrugated ties shall not be used on veneers greater than 30 ft (9.14 mm), or 38 ft (11.58 m) at a gable, in height.
	Sheet Metal	4 in. (101.6 mm) ~~[2]~~	Fastener: Minimum No. 10 screw(s) with 1 $^{3}/_{8}$ in. (34.9 mm) penetration into backing, or, where sheathing is present, into the structural member behind the sheathing. Exterior veneer exceeding 30 ft (9.1 m), or 38 ft (11.58 m) at a gable, in height above the vertical support shall be designed and detailed to provide for differential movement.
	Adjustable	6 in. (152 mm) ~~[2]~~	Fastener: Minimum No. 10 screw(s) with 1 $^{3}/_{8}$ in. (34.9 mm) penetration into backing, or, where sheathing is present, into the structural member behind the sheathing. Exterior veneer exceeding 30 ft (9.1 m), or 38 ft (11.58 m) at a gable, in height above the vertical support shall be designed and detailed to provide for differential movement.
Cold-formed Metal Light ~~Metal~~ Framing	Adjustable	6 in. (152 mm) ~~[2]~~	Fastener: Minimum ~~corrosion resistant~~ No. 10 screw(s) extending through the ~~steel~~ cold-formed metal framing a minimum of three exposed threads. ~~Steel~~Cold-formed metal framing shall be corrosion resistant and have a minimum base metal thickness of 0.043 in. (1.1 mm).~~.~~ Exterior veneer exceeding 30 ft (9.1 m), or 38 ft (11.58 m) at a gable, in height above the vertical support shall be designed and detailed to provide for differential movement.
Concrete	Adjustable	6 in. (152 mm)	~~Fastener:~~ Fasteners shall meet the pullout resistance requirements of Section 13.2.2.3.2.~~Minimum 3/16 in. (4.76 mm) screw(s) with 1.5 in. (38.1 mm) embedment .~~
Clay or Concrete Masonry	Adjustable, Unit Wire, or Joint Reinforcement	6 in. (152 mm)	~~Fastener:~~ Fasteners shall meet the pullout resistance requirements of Section 13.2.2.3.2.~~When required, m~~~~Minimum 3/16 in. (4.76 mm) screw(s) with 1.5 in. (38.1 mm) embedment~~ ~~[1]~~~~. Not applicable for joint reinforcement.~~

[1] Unit wire ties and joint reinforcement do not require fasteners.

~~[2] The cavity width shall be permitted to be increased by the specified thickness of the sheathing up to 5/8 in. (15.9 mm) for sheathing or veneer ties meeting the requirements of Section 13.2.2.3.3.~~

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13.2.2.3.1 Veneer shall be designed for a vertical application. Out-of-plane corbelling shall meet the requirements of Section 5.~~5~~6.2.

13.2.2.3.2 The pullout resistance of fasteners shall have a minimum design strength of 335 lb (1,490 N) or an allowable load of 200 lb (890 N). Fasteners listed and installed in accordance with Table 13.2.2.3 shall be deemed to comply.

13.2.2.3.3 The specified cavity width shall be from the face of the backing to the inside face of the veneer. The specified cavity width shall be permitted to be from the face of the sheathing to the inside face of the veneer if either of the following conditions is met:

(a) the bearing stress of the veneer tie on the sheathing from allowable stress level loads is less than the allowable bearing stress of the sheathing. ~~the sheathing has a minimum~~ Sheathing that has an allowable bearing stress of 100 psi (0.689 MPa) shall be deemed to comply~~, or~~

(b) the veneer ties penetrate the sheathing and directly contact the light frame backing~~have prongs with~~ and have a minimum allowable compressive strength of 200 lb (890N)~~ that penetrate the sheathing and directly contacts the light frame backing~~.

13.2.2.4 *Veneer ties* — Veneer ties shall comply with Table 13.2.2.4. Veneer ties with equivalent strength and stiffness shall be permitted. Veneer ties and fasteners (if present) shall be designed to provide no vertical support for the veneer. Veneer ties shall be fastened to, or embedded in, the backing for continuity of load path to the structural support of the veneer.

COMMENTARY

13.2.2.3.1 Although anchored veneer can be installed in non-vertical applications, the veneer ties, fasteners and support need to be engineered as these unique loading conditions are not considered in the prescriptive requirements. Designs that exceed the prescriptive corbeling limitations of Section 5.~~5~~6.2 would need to use modeling analysis method of Section 13.2.3.3.

13.2.2.3.2 The design strength of 335 lb (1,490 N) should be used with strength level loads, and the allowable load of 200 lb (890 N) should be used with allowable stress level loads.

Prior to the 2022 edition of this Code, 8d common nails were allowed for attaching veneer ties to wood light frame backing. Although there have been years of successful performance with common nails, ring-shank nails have a greater withdrawal resistance and provide improved performance (Klingner et al (2010)).

13.2.2.3.3 Determining the specified width of the cavity will depend on the type of backing, whether sheathing is present and its properties or whether the veneer tie~~ contains prongs~~. For masonry or concrete backings, the cavity width is from the face of the backing to the inside face of the veneer. For light frame backing, that may or may not have sheathing, the requirements in this section define how the cavity should be measured when sheathing is present. The deemed to comply allowable bearing value of 100 psi (0.689 MPa)~~This value~~ would be met by typical OSB and plywood sheathing, and some gypsum sheathings. If the allowable bearing stress of the sheathing is less than 100 psi (0.689 MPa) such as with some foam sheathings, the veneer tie would need to ~~have prongs or another means of~~ transfer~~ring~~ the load through the sheathing to the backing or the bearing stress on the sheathing would need to be checked. ~~Penetration into sheathing alone cannot provide the pullout strength required to use the prescriptive requirements for anchored masonry veneer.~~

13.2.2.4 *Veneer ties* — Each veneer tie type has physical requirements that must be met. TMS 602 Article 3.4 ~~E~~ D has minimum embedment requirements to ensure load resistance against push-through or pull-out of the mortar joint. For proper performance, all of the fasteners required by the manufacturer's recommendations must be used. This is true of slotted adjustable veneer ties and other veneer ties that require two fasteners. Examples of some common veneer ties are shown in Figure CC-13.2.5.

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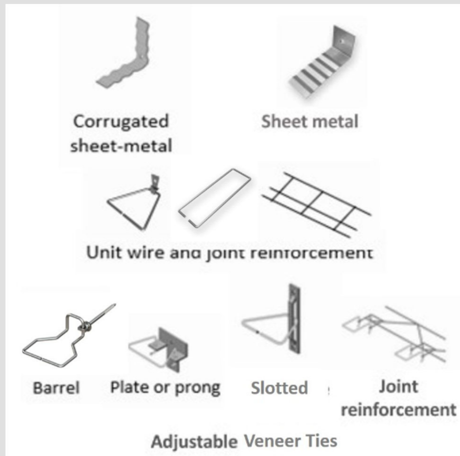

Figure CC-13.2-5 — Examples of veneer ties

Veneer ties typically allow for movement in the plane of the wall, but resist movement perpendicular to the veneer. The mechanical play (clearance between adjustable parts) in adjustable veneer ties and the stiffness of the tie influence load transfer between the veneer and the backing.

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Veneer ties of wire with drips are not permitted because of their reduced load capacity.

The term "offset" in Table 13.2.2.4 refers to the vertical distance between a wire eye and the horizontal leg of a bent wire tie inserted into that eye, or the clear vertical distance (gap) between functionally similar components of an adjustable veneer tie.

For cavity widths that exceed 4 in. (102 mm), adjustable veneer ties must meet additional requirements. A cross section through a wall with such an adjustable tie is shown in Figure CC-13.2-6. The distance from the inside face of the veneer to the end of the adjustable part is limited to 2 in. (51 mm).

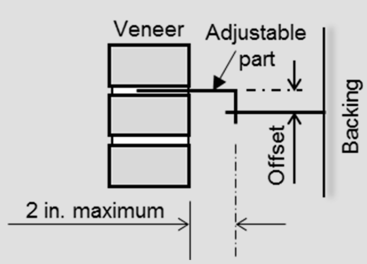

Figure CC-13.2-6 — Adjustable veneer tie for cavity widths that exceed 4 in. (102 mm)

Table 13.2.2.5: Veneer Tie Requirements

Tie type	Requirements
Corrugated sheet-metal	1) Minimum 7/8 in. (22.2 mm) wide, base metal thickness minimum of 0.03 in. (0.8 mm). 2) Corrugation wavelength: 0.3 to 0.5 in. (7.6 to 12.7 mm). 3) Corrugation amplitude: 0.06 to 0.10 in. (1.5 to 2.5 mm).
Sheet-metal	1) Minimum ⅞ in. (22.2 mm) wide, base metal thickness minimum of 0.06 in. (1.5 mm). 2) Shall have either: a. Corrugations with wavelength of 0.3 to 0.5 in. (7.6 to 12.7 mm) and amplitude of 0.06 to 0.10 in. (1.5 to 2.5 mm), or b. Bent, notched, or punched to provide equivalent performance in pull-out or push-through.
Unit wire	1) Minimum W1.7 (MW11) wire ~~with and have ends bent to form an extension from the bend~~ where the length of the wire that is parallel to and within the veneer is at least 2 in. (50.8 mm) long~~.~~ within the veneer for Z-ties. ~~1)~~2) Minimum W1.7 (MW11) wire with the total length of the wire within the veneer is at least 2 in. (50.8 mm) long for box and triangular unit ties. ~~2)~~3) Drips are not permitted.
	When cavity width exceeds 4 in. (101.6 mm): wires shall be minimum W2.8 (MW18).
Joint reinforcement	1) Ladder-type, truss-type or tab-type joint reinforcement is permitted. Truss-type joint reinforcement across the cavity is not permitted. 2) Longitudinal wires: minimum W1.7 (MW11) size. 3) Cross wires: minimum W1.7 (MW11) wire and spaced at maximum of 16 in. (406 mm) o.c. 4) Drips are not permitted in cross wires or tabs.
	When cavity width exceeds 4 in. (101.6 mm): cross and longitudinal wires shall be minimum W2.8 (MW18).
Adjustable	1) Sheet metal components shall conform to sheet-metal tie requirements. 2) Wire components shall conform to unit wire tie requirements. 3) Adjustable veneer ties with joint reinforcement shall also conform to joint reinforcement tie requirements. 4) Maximum clearance between connected parts of 1/16 in. (1.6 mm). 5) Detailed to prevent disengagement. 6) One or more pintle legs of minimum W2.8 (MW18), have two wires embedded in the veneer, and have a vertical wire offset not exceeding 1.25 in. (31.8 mm).
	1) Part of veneer tie attached to backing: a. For concrete, masonry, wood light framing or cold-formed metal light framing: (1) Barrel with minimum outside diameter of 3/16 in. (4.76 mm) and composed of solid metal. (2) Plate or prong at least 0.074 in. (1.88 mm) thick and 1-1/4 in. (31.8 mm) wide. b. For masonry backing: a tab or two eyes formed of minimum W2.8 (MW18) wire welded to joint reinforcement. 2) Where cavity width exceeds 4 in. (101.6 mm) a. Adjustable part: (1) Two or more pintle legs of minimum W2.8 (MW18) wire. (2) Distance from inside face of veneer to end of adjustable part: maximum 2 in. (51 mm).

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13.2.2.5 *Prescriptive veneer tie spacing* — The maximum tributary area and maximum spacing of a veneer tie shall be in accordance with Table 13.2.2.5. The maximum spacing shall apply to both vertical and horizontal spacing of the veneer ties.

Veneer ties shall be located within 16 in. (406 mm) of supported edges and within 12 in. (305 mm) of unsupported edges, openings, and movement joints. The distance from the top of the veneer to the first row of veneer ties shall not exceed one-half the maximum spacing given in Table 13.2.2.5.

Table 13.2.2.5: Prescriptive anchored veneer tie spacing [1]

	Basic	Enhanced
Maximum tributary area per tie	2.67 ft^2 (0.248 m^2)	1.78 ft^2 (0.165 m^2)
Maximum spacing	24 in. (610 mm)	16 in. (406 mm)

[1] See Table 13.2.1.1 for when Basic and Enhanced are required.

13.2.3 *Engineered design of anchored masonry veneer* — The engineered design of anchored veneer shall comply with the requirements of Section 13.2.3.1 and either Section 13.2.3.2 or Section 13.2.3.3. The veneer is not subject to the allowable flexural tensile stress provisions of Section 8.2.4.2 or the modulus of rupture provisions of Section 9.1.9.1~~2~~. Units shall comply with Section 13.2.2.1~~ or TMS Article 2.3 C~~, or alternately, testing shall be performed to determine necessary material properties.

13.2.3.1 *Strength and stiffness of veneer ties*

13.2.3.1.1 *Deemed to comply strength and stiffness* — The ~~designer shall be permitted to utilize the~~ strength and stiffness values of Table 13.2.3.1.1 shall be permitted to be utilized for veneer ties that meet the requirements of Table 13.2.2.4.

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13.2.2.5 *Prescriptive veneer tie spacing* — Veneer ties are permitted to be placed 16 in. (406 mm) from supported edges, including the base of the veneer, shelf angles, and lintels to accommodate flashing placement while still providing a connection from the veneer to the backing. Veneer ties need to be placed within 12 in. (305 mm) of any unsupported edge, including at openings and movement joints. Movement joints can be both horizontal and vertical and typically consist of control joints, expansion joints, construction joints, and isolation joints.

The top row of veneer ties has been shown to be critical to the performance of the veneer (Page et al (1996); Yi et al (2003); Reneckis and LaFave (2010)). Therefore, the spacing at the top of the veneer is limited to one-half the maximum spacing.

13.2.3 *Engineered design of anchored masonry veneer* — Engineered design is used when the Architect/Engineer wants to exceed the limitations of Section 13.2.2 or when the veneer does not qualify to be designed under the prescriptive requirements. Engineered design options include the tributary area method (Section 13.2.3.2) or modeling analysis method (Section 13.2.3.3).

13.2.3.1 *Strength and stiffness of veneer ties*

13.2.3.1.1 *Deemed to comply strength and stiffness* — Strength and stiffness values from Table 13.2.3.1.1 can be used. The intent is that these values be used on an interim basis for design until additional test data on veneer ties becomes available.

Design strengths implicitly include the strength-reduction factor and can be directly compared with strength level loads.

Values for corrugated sheet-metal ties were obtained from Choi and LaFave (2004). Values for other ties were based on Drysdale and Wilson (1989), Porter (1990), and data from veneer tie manufacturers.

The veneer ties listed in Table 13.2.3.1.1 are shown in Figure CC-13.2-5. Insufficient data was available for other tie types to provide strength and stiffness values.

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Table 13.2.3.1: Veneer t~~T~~ie axial strength and stiffness values

Veneer Tie	Design Strength	Allowable Load	Stiffness
Corrugated sheet-metal	125 lb (556 N)	75 lb (334 N)	500 lb/in. (87.6 N/mm)
Adjustable – slotted	330 lb (1468 N)	200 lb (890 N)	3000 lb/in. (525 N/mm)
Adjustable - two leg pintle	210 lb (934 N)	125 lb (556 N)	2500 lb/in. (438 N/mm)
Unit wire veneer ties and joint reinforcement	210 lb (934 N)	125 lb (556 N)	20000 lb/in. (3500 N/mm)

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13.2.3.1.2 *Determination of strength and stiffness by test* — The strength and stiffness of veneer ties determined by test shall be in accordance with the following:

(a) All components of a veneer tie shall be tested, except when an engineering analysis can be performed to reliably predict the connector's behavior and strength. The analysis shall be based on established engineering principles and the properties of the material forming the component and shall consider the effects of the interfacing components.

(b) In testing, the structural backing, edge distances, spacings, cavity widths, fastener type, embedment/anchorage, mechanical play, sheathing, insulation, restraints, masonry materials and construction, and all other materials, components, assemblies, and configurations shall be representative of those that will be used in service.

(c) A minimum of five tests shall be conducted.

(d) For adjustable veneer ties, the ultimate strength shall not be taken greater than 1.5 multiplied by the strength determined at the configuration that results in the lowest nominal strength.

(e) For strength design, the design strength shall be 0.5 multiplied by the average ultimate strength determined by test. The design strength is permitted

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13.2.3.1.2 *Determination of strength and stiffness by test* — This test method is used to determine the strength and stiffness of veneer ties. Tests such as ASTM E754 and CSA A370 determine other properties of veneer ties.

(a) The testing program should capture all variables that may influence the performance of the veneer tie assembly. Where the performance of one or more components of the veneer tie assembly has already been established, for example, the method of connecting the tie to the backing, such variables may be considered as part of the analytical analysis of the veneer tie assembly performance.

(b) If components of the masonry assembly do not comply with Section 13.2.1, testing is necessary. The objective of the testing is to determine that the veneer tie has adequate strength and stiffness and that the method(s) by which the veneer tie transfers loads from the veneer to the backing are adequate for the intended use.

(c) In accordance with Section 13.2.3.1.2(e), higher design strengths are permitted when additional tests are conducted.

(d) The strength and stiffness of adjustable veneer ties are generally correlated. As the stiffness of the veneer tie decreases, the load distribution changes, and the maximum tie force generally decreases. Thus, as the stiffness of the veneer tie decreases, the strength decreases, but the demand also decreases. For adjustable veneer ties, the ultimate load and stiffness can be taken as the ultimate load and stiffness with the veneer tie halfway between the location of minimum and maximum stiffness. Using the strength and stiffness at a location halfway between maximum and minimum stiffness is a reasonable way to account for the proportional relationship between strength and stiffness and to not be excessively conservative in design. An upper bound of 1.5 multiplied by the minimum ultimate strength is used to guard against too low of a strength at the weakest configuration.

(e) The factor for determining design strength from the average ultimate load from tests includes inherent variability (assumed to be a coefficient of variation of approximately 0.25), statistical uncertainty, and a

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to be 0.6 multiplied by the average ultimate strength if ten or more tests are conducted.

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(f) The allowable force is 0.6 multiplied by the design strength determined in accordance with Section 13.2.3.1.2(e).

(g) The stiffness of the veneer tie shall be determined as the secant stiffness between 10 lb (44.5 N) and 40% of ultimate load.

13.2.3.2 *Tributary area method* — The tributary area method shall be permitted for veneers meeting the requirements of Section 13.2.2.1 and 13.2.2.3.1 and having a thickness of 5 in. (127 mm) or less.

The strength level force in each veneer tie shall be determined as:

$2p_uA_t$ when $k_{tie} \leq 2500$ lb/in. (350 N/mm)

$2.5p_uA_t$ when 2500 lb/in. (350 N/mm) < $k_{tie} \leq$ 5000 lb/in. (876 N/mm)

$3p_uA_t$ when 5000 lb/in. (876 N/mm) < $k_{tie} \leq$ 8000 lb/in. (1401 N/mm)

$4p_uA_t$ when k_{tie} > 8000 lb/in. (1401 N/mm)

Allowable stress level forces in each veneer tie shall be determined by replacing p_u with p_{allow}.

13.2.3.3 *Modeling analysis method* — For the modeling analysis method, the distribution of forces from the veneer to the veneer ties and the backing shall be modeled based upon principles of mechanics. Such analyses shall include the relative stiffness of the veneer, the veneer tie, and the backing. Locations in the veneer where the flexural tensile stress exceeds the modulus of rupture of Section 9.1.9.1~~2~~ shall be considered to be cracked and shall be permitted to be modeled as a hinge.

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strength-reduction factor (assumed to be 0.9). The nominal strength is determined as the 75th percent confidence level of the 5th-percentile value. The higher permitted design strength for ten tests reflects the reduction in statistical uncertainty. Design strengths can be directly compared with strength level loads.

(f) No commentary.

(g) The lower bound on stiffness is to accommodate 'seating' within the tested tie assembly. The upper bound is felt to be a reasonable limit on the tie assembly working load.

13.2.3.2 *Tributary area method* — Tie stiffness has the largest effect on tie force, although other parameters including backing stiffness, veneer stiffness, and veneer cracking influence the load distribution. Lower tie stiffness results in a more uniform distribution of ties forces, while higher tie stiffness results in a much more non-uniform distribution of tie forces. The more the veneer cracks, the more uniform the distribution of tie forces. Stiffer backings also result in more uniform tie forces.

The tributary area method is limited to veneers of 5 in. (127 mm) or less. Thicker, and, therefore, stiffer veneers will result in increased nonlinear distribution of veneer tie forces, and the tie forces determined from the tributary area method may be unconservative.

The value for p_u (strength level) or p_{allow} (allowable stress level) is the out-of-plane load on the veneer ties, either negative or positive.

The design values were chosen based on extensive analyses of anchored veneer systems that considered various veneer tie stiffness values, various backing stiffness values, different heights, different mortar types, and solid and hollow veneer units for simple spans (Hochwalt et al (2019)). Multi-span backing, backing with cantilevers, and backing interrupted with openings were not considered.

13.2.3.3 *Modeling analysis method* — The force in a veneer tie will depend on the relative stiffness of the backing, veneer, and tie, as well as the extent of cracking in the veneer. Wall systems with stiff backings, flexible ties, and lower cracking strength of veneer will generally result in a more uniform distribution of the load to the veneer ties, and lower peak tie forces.

Even though the stiffness of the veneer is not considered in the design of the backing, the stiffness of the veneer will affect the forces in the veneer ties, and the veneer stiffness needs to be included in the analysis to determine veneer tie forces.

Further information on the analysis of brick veneer wall systems is in Brown and Arumula (1982), Kelly et al (1990), and Yi et al (2003).

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Because there are no constraints on the use of the Modeling Analysis method for anchored veneers, the design requires a comprehensive analysis and assessment of the entire assembly. This includes, but is not limited to, load magnitude and load distribution, force transfer through the system, potential second order effects, absolute and relative displacements, local deformation of the backing, mechanical play of the veneer tie, differential movements, and possible implications of long-term movements. Although using the prescriptive design requirements doesn't require properties such as the specified compressive strength, f'_m, an engineering analysis may require such a property as well as other design values found in other parts of this ~~code~~Code.

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13.3 — Adhered veneer

13.3.1 *General requirements for adhered veneer*

13.3.1.1 *Scope* — This section provides requirements for the design of adhered veneer as defined in Section 2.2. Adhered veneer shall comply with the requirements of either Section 13.3.2 or 13.3.3.

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13.3 — Adhered veneer

Adhered veneer differs from anchored veneer in its means of attachment. Positive (compressive) out-of-plane loads are transferred directly to the masonry or concrete backing when directly bonded to the backing, or for light frame backing, through the sheathing to the light frame backing elements. Negative (tensile) out-of-plane loads are transferred directly to the masonry or concrete backing when directly bonded to the backing, or for light frame backing, through the fasteners to the light frame elements. Vertical (gravity) loads are transferred directly to the backing when fasteners are not present, or through the fasteners to the backing when fasteners are present. When sheathing is present behind an adhered veneer, other than providing a load path for compressive out-of-plane loads into the backing, it is assumed to provide no contribution to the strength or stiffness of the adhered veneer assembly or fasteners.

The designer should provide for proper means of bonding units to the backing, attachment of the lath and scratch coat or cement backer unit to the backing~~structure~~, control curvature of the backing, account for differential movement, consider freeze-thaw cycling, water penetration, air leakage, and vapor diffusion. There are proprietary systems that can demonstrate compliance with this section. Manufacturer documentation including submittals should be consulted and referenced as required in TMS 602 Article 1.5.

The cross-section of a typical adhered veneer wall where the backing is either wood or metal studs is shown in Figure CC-13.3-1.

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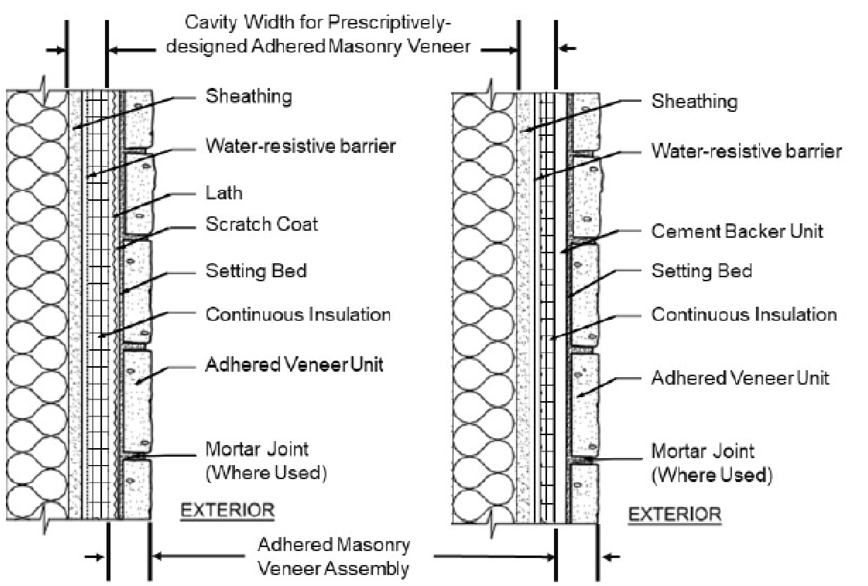

(a) with lath (b) with cement backer unit

Figure CC-13.3-1 — Cross-section of typical adhered masonry veneer supported by light frame backing

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13.3.1.2 *Out-of-plane deflection* — Out-of-plane deflection of the backing shall be limited to $h_b/360$ under application of 0.42 times the strength level wind load, and $h_b/150$ under application of the strength level seismic load.

13.3.1.2 *Out-of-plane deflection* — The out-of-plane deflection limitation is both to limit veneer cracking and to provide out-of-plane stability under the eccentric load from the adhered veneer.

Cracking of adhered veneer may be more related to curvature than deflection limits. For a deflection limit of $h_b/360$, the maximum curvature is $9.6/(360h_b)$. Therefore, curvature increases with decreasing span. More stringent deflection limits may be appropriate for smaller spans. The deflection criteria differ for wind and seismic because different levels of load are used. A service load is used for wind while a strength level load is used for seismic. See Commentary Section 13.2.1.5 for additional information.

13.3.1.3 *Water penetration resistance* — Exterior adhered veneer wall systems shall be designed and detailed to resist water penetration~~ into the building interior~~.

13.3.1.3 *Water penetration resistance* — Water penetration through the exterior veneer is expected~~. The~~; however, wall system must be designed and constructed to prevent water from breaching the building envelope~~entering the building~~. Information and references on designing and detailing for water penetration resistance are located in Section 13.1.2.1.

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Since water penetration is a critical issue for adhered masonry veneer, consideration should be given to appropriate drainage layers within the adhered veneer

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13.3.2 *Prescriptive design of adhered masonry veneer*

13.3.2.1 *Permitted units* — Prescriptively-designed adhered veneer shall be constructed of units complying with ASTM C1088, ASTM C1364, ASTM C1670/C1670M, or ASTM C1877. Units complying with ASTM C73 or TMS 602 Article 2.3 C shall be permitted provided the bond developed between adhered veneer units and backing has a shear strength of at least 50 psi (345 kPa) based on gross unit bonded area when tested in a laboratory in accordance with ASTM C482 using the specified unit, mortar and substrate.

13.3.2.2 *Unit limitations* — Units for prescriptively-designed adhered veneer shall comply with the following:

(a) The average thickness of adhered masonry veneer units shall not exceed 2.625 in. (67 mm).

(b) The bonded surface area of each adhered masonry veneer unit shall not exceed 720 in.^{2} (0.465 m^2). Units having a bonded surface area greater than 360 in.^{2} (0.232 m^2) shall have an installation procedure approved by the ~~Architect/Engineer~~Licensed Design Professional.

(c) The weight of adhered masonry veneer units shall not exceed 30 psf (146.5 kg/m^2).

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system. Use of adhered masonry veneers with tight-fit joints (joints between adhered veneer units that are not purposely filled with mortar), also referred to as drystack veneer, should be carefully considered in wet climates that include freeze thaw conditions and should closely follow the installation requirements in TMS 602 Article 3.3 C.

13.3.2 *Prescriptive design of adhered masonry veneer*

Adhered veneers are bonded to either:

(a) a masonry or concrete backing or
(b) a scratch coat and lath or
(c) cement backer unit that is fastened to masonry, concrete, or light frame backing.

13.3.2.1 *Permitted units* — The design strengths are based on bond between the unit and the mortar, and the backing and the mortar. The strength of other components in the system also needs to be considered. The strength could be controlled ~~by the backing~~within the assembly, ~~such as~~ a shear failure in a cement backer unit or elsewhere within other layers within the system. Field quality assurance testing of adhered masonry veneer may use ASTM C1823 to confirm compliance.

ASTM C482 is a laboratory test method to qualify that an adhered masonry unit develops adequate bond strength at its bonding surface with a specified adhesive over a specified substrate. The method is often adapted to include materials that will be used in construction. ASTM C482 is not intended to evaluate the bond strength between various combinations of masonry units, setting bed mortar, membranes, and backings. Testing procedures should be modified for the materials used and testing conditions.

ASTM C1823 is a test method that is used in the field to measure the shear bond strength in situ. This test method includes failure modes beyond the normal unit and mortar bond, therefore failures that occur within the units or within the substate may not be appropriate for qualifying materials (Dillon and Dalrymple (2021)).

13.3.2.2 *Unit limitations* — The ~~weight and density~~ unit limitations are imposed to reduce the difficulties of handling and installing large units and to help ensure good bond. Units having a bonded surface area greater than 360 in.^{2} (0.232 m^2) may be more susceptible to installation problems due to the difficulty in obtaining proper and complete bond. Consideration should be given to back buttering the unit or using multiple workers for the installation of each unit.

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13.3.2.3 ~~*Scratch*~~ *Mortar requirements for scratch coat, setting bed, and* ~~*jointing mortar requirements*~~*joints between units*

(a) Scratch coat shall comply with TMS 602 Article 2.1 A, Type M or S or Article 2.1 B.

(b) Setting bed mortar shall comply with TMS 602 Article 2.1 B.

(c) ~~Jointing mortar~~ Mortar between units shall comply with TMS 602 Article 2.1 A, Type S or N, or Article 2.1 B.

13.3.2.4 *Installation requirements* — Lath and scratch coat shall not be required when adhered masonry veneer units are applied directly to concrete, ~~concrete~~ unglazed clay or concrete masonry, or cement backer units free of coatings, debris, membranes, or similar materials that would inhibit bond to ~~the backing~~those surfaces.

13.3.2.5 *General requirements* — Prescriptively-designed adhered masonry shall comply with the following:

(a) The distance from the exterior surface of the adhered masonry veneer units to the interior surface of the scratch coat or cement backer unit shall not exceed 4.625 in. (117 mm).

(b) The height above grade plane shall not exceed 60 ft (18.3 m).

(c) Backing to which adhered veneer masonry units are installed shall be in a vertical application.

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13.3.2.3 ~~*Scratch*~~*Mortar requirements for scratch coat, setting bed, and* ~~*jointing mortar requirements*~~ joints between units — Prescriptive design requires the use of polymer modified mortar for the setting bed. When installed properly, polymer modified mortars typically have superior bond strength. However, the use of polymer modified mortars will not compensate for unsuitable or poorly prepared substrates, substrates having contaminants, improper mortar preparation or poor workmanship including partially filled setting beds. Scratch coats and ~~pointing~~ mortar joints between units use either traditional mortar or polymer modified mortar.

13.3.2.4 *Installation requirements* — Installation of adhered masonry veneer units must comply with TMS 602. Lath and scratch coat are not required when adhered masonry veneer units are applied directly to ~~certain backings (~~concrete, concrete masonry, or cement backer units~~)~~ ~~due to~~ which provide adequate bond. Differential movement between adhered veneer units and the backing should be considered as their incompatibility may result in cracks or debonding.

When concrete, clay masonry, or concrete masonry walls are smooth, have a glazed coating, or where good bond cannot be achieved, adhered veneer systems should be installed over lath. The surfaces intended to receive adhered units must have a rough texture to ensure good mortar bond. ICRI Technical Guideline 310.2 (ICRI 2013) provides information on ~~concrete~~ surface preparation, including information on Concrete Surface Profile, a standardized method to measure concrete surface roughness. A Concrete Surface Profile equal to or greater than 2 is usually acceptable for the installation of adhered veneer over concrete and masonry assemblies but verification for specific project conditions may be required. When testing is warranted due to surface texture of the substrate or the presence of a membrane or coating that may inhibit bond, the procedures of Section 13.3.3 should be followed.

13.3.2.5 *General requirements*

(a) The thickness limit from the back of the scratch coat is to limit the eccentric load applied to the system.

(b) The height restriction for prescriptively designed adhered veneer is used to limit wind pressures and based on empirical evidence.

(c) Although adhered veneer can be installed in a non-vertical application (i.e. soffit), the fastening system would need to be engineered as these unique loading conditions are not considered in the prescriptive tables of Section 13.3.2.5.

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(d) Design and detailing shall consider differential movement between the veneer and the backing.

(e) ~~The prescriptive design of adhered masonry veneer shall comply with the requirements of either Table 13.3.2.5 or Table 13.3.2.6 or shall be directly applied to concrete or masonry backing~~Adhered masonry veneer shall comply with the requirements of Table 13.3.2.5, Table 13.3.2.6, or shall be directly applied to concrete or masonry backing.

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(d) When directly bonding adhered masonry units to a backing of dissimilar material, for example, thin clay units adhered to a concrete masonry backing, the potential for differential movement and veneer delamination or distress increases significantly.

Accommodating the potential for differential movement is required and may be accomplished by installing a bond break and lath between the veneer and backing, a drainage plane, or allowing the backing to sufficiently cure before the adhered veneer is installed. When bonding thin clay units to a concrete masonry or concrete backing, industry guidelines have recommended allowing the concrete backing to cure for a minimum of 45 days.

(e) The critical load path when attaching an adhered veneer to light frame backing is through the fasteners used to install the lath over the backing. These fasteners are subjected to axial forces resulting from out-of-plane wind and seismic loads and lateral shearing forces from gravity and seismic loads. Tables 13.3.2.5 and 13.3.2.6 provide maximum fastener spacing requirements for common fastener types. Given the wide array of fastener types available, however, each table also provides a minimum withdrawal and lateral strength that must be satisfied where a different fastener is selected. These withdrawal and lateral strengths must account for the reduced embedment depth of the fastener due to nonstructural materials such as insulation within the assembly cavity. See Figure CC-13.3-1 for determination of cavity width. Fasteners are assumed to be partially embedded into their substrate due to the

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(f) *Sheathing* — Sheathing is required over light frame backing receiving an adhered veneer assembly.

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presence of the cavity. The withdrawal and lateral strengths of partially embedded fasteners is derived from the Wood Handbook (FPL (2010)). Table 13.3.2.5 assumes a specific gravity value of 0.40 for the wood light frame backing and no strength adjustments for loading duration, wet service conditions, or extreme temperatures.

When fastening lath to light frame backing, prescriptive installation methods have limited the spacing of fasteners along the length of the studs to 7 in. (178 mm). While the use of stiffer or stronger fasteners may allow the spacing of fasteners to be increased when designed in accordance with Section 13.3.3, the prescriptive fastener values of Tables 13.3.2.5 and 13.3.2.6 maintain this historical limit on fastener spacing.

The prescriptive design options for adhered veneers were verified for both out-of-plane wind and seismic forces; with wind producing the controlling design load. While design wind pressures vary at different surfaces over a building, the largest design pressure was used in establishing the prescriptive criteria of this chapter. The design of a more economical fastener is possible at discrete zones across a building's surface where lower design wind pressures exist. However, in accordance with the requirements of ASCE/SEI 7, corner negative pressures are required to be applied to the entire height of the building. The design of the fasteners for vertical gravity loads, accounting for seismic, is often controlled by the flexural strength and stiffness of the fastener, particularly as the weight of the veneer assembly or thickness of the cavity increases. Therefore, common nails are specified because they have greater flexural strength and stiffness than other nail types. For both out-of-plane and gravity loads, conservative design values for the fastener withdrawal and shear strength were assumed accounting for the reduced fastener embedment depth due to the presence of sheathing, air space, insulation, or other materials within the cavity. ~~Table 13.3.2.5 assumes a conservative specific gravity value of 0.40 for the wood light frame backing and no strength adjustments for loading duration, wet service conditions, or extreme temperatures.~~ Table 13.3.2.6 requires a backing of light steel framing having a minimum thickness of 16 gauge (1.5 mm) and a minimum yield strength of 50,000 psi (345 MPa) or larger. A more economical design could be achieved using the procedures of Section 13.3.3.

(f) Sheathing is required over light frame backing receiving an adhered veneer assembly in accordance with TMS 602 Article ~~3.3 C.1~~3.3 D.1. Adhered veneer assemblies are not intended to span between framing members and thus require the presence of sheathing to

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(g) *Assembly weight* — The weight of the prescriptively-designed adhered veneer assembly shall not exceed a specified weight of 50 psf (2.4 kPa). The assembly weight shall include the weight of the units, setting bed mortar, scratch coat, lath, and other materials attached to the backing, where present.

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perform as intended. See also Commentary Section 13.3.3 for additional information.

(gf) The limitation on installed weight is to limit the force under seismic and gravity loads.

Table 13.3.2.5: Veneer Fastener Spacing Along Backing, p_{veneer} ≤ 60 psf (2.87 kPa) (Wood Light Framing with 16 in. (406 mm) Spacing) [1, 2, 3, 4,5]

Fastener Type	Adhered Veneer Assembly Installed Weight, psf (kPa)							
	10 (0.48)	20 (0.96)	25 (1.2)	30 (1.4)	35 (1.7)	40 (1.9)	45 (2.2)	50 (2.4)
Cavity Width ≤ 0.5 in. (12.7 mm)								
6d Nail 140 / 60 (0.62 / 0.27) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	6.5 in. (165 mm)	5.8 in. (148 mm)	5.3 in. (135 mm)	4.8 in. (123 mm)	4.5 in. (114 mm)
10d Nail 300 / 90 (1.33 / 0.40) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)
0.5 in. (12.7 mm) < Cavity Width ≤ 1.0 in. (25.4 mm)								
10d Nail 240 / 90 (1.07 / 0.40) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	6.8 in. (174 mm)	6.1 in. (156 mm)	5.5 in. (140 mm)
16d Nail 330 / 100 (1.47 / 0.44) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)
1.0 in. (25.4 mm) < Cavity Width ≤ 1.5 in. (38.1 mm)								
10d Nail 180 / 90 (0.80 / 0.40) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	6.2 in. (157 mm)	5.3 in. (136 mm)	4.6 in. (117 mm)	4.1 in. (105 mm)	3.7 in. (95 mm)
16d Nail 260 / 100 (1.16 / 0.44) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	6.1 in. (156 mm)	5.4 in. (138 mm)	4.9 in. (124 mm)
1.5 in. (38.1 mm) < Cavity Width ≤ 2.0 in. (50.8 mm)								
10d Nail 120 / 90 (0.53 / 0.40) [6]	7.0 in. (177 mm)	6.8 in. (174 mm)	5.6 in. (142 mm)	4.6 in. (117 mm)	4.0 in. (102 mm)	3.5 in. (89 mm)	3.1 in. (79 mm)	2.8 in. (71 mm)
16d Nail 190 / 100 (0.85 / 0.44) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	6.1 in. (156 mm)	5.2 in. (133 mm)	4.6 in. (117 mm)	4.0 in. (104 mm)	3.6 in. (93 mm)
2.0 in. (50.8 mm) < Cavity Width ≤ 2.5 in. (63.5 mm)								
16d Nail 130 / 100 (0.58 / 0.44) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	5.9 in. (149 mm)	4.9 in. (124 mm)	4.2 in. (107 mm)	3.6 in. (93 mm)	3.2 in. (83 mm)	2.9 in. (74 mm)
20d Nail 230 / 130 (1.02 / 0.58) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	6.1 in. (156 mm)	5.4 in. (138 mm)	4.9 in. (124 mm)

[1] Cavity width measured from face of stud to back of veneer assembly.
[2] Fastener spacing for SDC D, E, and F shall be limited to 80% of the listed values.
[3] Linear interpolation shall not be permitted.
[4] Fastener placement tolerance according to TMS 602 Article 3.4 F.
[5] The minimum specific gravity for the wood backing shall be 0.40.
[6] Equivalent diameter fastener strength of common nails with minimum withdrawal strength and lateral strength shown, respectively (lb on first line and kN on second line).

Table 13.3.2.6: Veneer Fastener Spacing Along Backing, p_{veneer} ≤ 60 psf (2.87 kPa) (16 ga. (1.5 mm) Cold-formed Metal Framing with 16 in. (406 mm) Spacing) [1, 2, 3, 4, 5]

Fastener Type	Adhered Veneer Assembly Installed Weight, psf (kPa)							
	10 (0.48)	20 (0.96)	25 (1.2)	30 (1.4)	35 (1.7)	40 (1.9)	45 (2.2)	50 (2.4)
Cavity Width ≤ 0.5 in. (12.7 mm)								
#8 Screw 375 / 375 (1.67/1.67) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)
#10 Screw 375 / 375 (1.67/1.67) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)
0.5 in. (12.7 mm) < Cavity Width ≤ 1.0 in. (25.4 mm)								
#8 Screw 375 / 375 (1.67/1.67) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	6.5 in. (165 mm)	5.7 in. (146 mm)	5.1 in. (131 mm)	4.6 in. (117 mm)	4.1 in. (105 mm)
#10 Screw 375 / 375 (1.67/1.67) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	6.4 in. (164 mm)
1.0 in. (25.4 mm) < Cavity Width ≤ 1.5 in. (38.1 mm)								
#8 Screw 375 / 375 (1.67/1.67) [6]	7.0 in. (177 mm)	6.5 in. (165 mm)	5.4 in. (138 mm)	4.6 in. (118 mm)	4.0 in. (102 mm)	3.5 in. (90 mm)	3.1 in. (80 mm)	2.8 in. (72 mm)
#10 Screw 375 / 375 (1.67/1.67) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	6.2 in. (159 mm)	5.5 in. (139 mm)	4.8 in. (124 mm)	4.4 in. (111 mm)
1.5 in. (38.1 mm) < Cavity Width ≤ 2.0 in. (50.8 mm)								
#8 Screw 375 / 375 (1.67/1.67) [6]	7.0 in. (177 mm)	5.1 in. (131 mm)	4.5 in. (108 mm)	3.5 in. (90 mm)	3.0 in. (77 mm)	2.6 in. (67 mm)	2.3 in. (60 mm)	2.1 in. (54 mm)
#10 Screw 375 / 375 (1.67/1.67) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	6.6 in. (167 mm)	5.5 in. (139 mm)	4.7 in. (119 mm)	4.1 in. (104 mm)	3.6 in. (93 mm)	3.3 in. (83 mm)
2.0 in. (50.8 mm) < Cavity Width ≤ 2.5 in. (63.5 mm)								
#10 Screw 375 / 375 (1.67/1.67) [6]	7.0 in. (177 mm)	6.6 in. (167 mm)	5.2 in. (134 mm)	4.4 in. (111 mm)	3.7 in. (95 mm)	3.3 in. (83 mm)	2.9 in. (74 mm)	2.6 in. (67 mm)
#12 Screw 375 / 375 (1.67/1.67) [6]	7.0 in. (177 mm)	7.0 in. (177 mm)	7.0 in. (177 mm)	6.4 in. (164 mm)	5.5 in. (141 mm)	4.8 in. (123 mm)	4.3 in. (109 mm)	3.8 in. (98 mm)

[1] Cold-formed metal framing minimum strength, 50 ksi (345 MPa).
[2] Cavity width measured from face of stud to back of veneer assembly.
[3] Fastener spacing for SDC D, E, and F shall be limited to 80% of the listed values.
[4] Linear interpolation shall not be permitted.
[5] Fastener placement tolerance according to TMS 602 Article 3.4 F.
[6] Equivalent diameter fastener strength with minimum withdrawal strength and lateral strength shown, respectively (lb on first line and kN on second line).

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13.3.3 *Engineered design of adhered masonry veneer* — The engineered design of adhered veneer shall satisfy the following conditions:

(a) Units shall comply with Section 13.3.2.1, or alternately, testing shall be performed to determine necessary material properties.

(b) Loads shall be distributed through the veneer to the backing using principles of mechanics.

(c) The vertical deflection of the veneer assembly shall be limited to 1/8 in. (3 mm) under strength level dead and seismic loads.

(d) The veneer shall not be subjected to the flexural tensile stress provisions of Section 8.2.4.2 or the nominal modulus of rupture provisions of Section 9.1.9.1~~2~~.

(e) Installation of adhered masonry veneer shall comply with TMS 602; otherwise, the specific installation procedures and materials shall be tested to determine appropriate design properties.

(f) When installation of adhered masonry veneer complies with TMS 602, the flexural tension design strength of the adhered veneer assembly components shall be permitted to be assumed to be 100 psi (689 kPa) and design shear strength of the adhered veneer assembly components shall be permitted to be assumed to be 50 psi (345 kPa) . For allowable stress design, the allowable flexural tension stress shall be permitted to be assumed to be 60 psi (414 kPa) and allowable shear stress shall be permitted to be assumed to be 30 psi (207 kPa).

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13.3.3 *Engineered design of adhered masonry veneer* — The intent of Section 13.3.3 is to permit the designer to use alternative unit thicknesses, areas, installation techniques, and units for adhered veneer. The designer should provide for proper means of bonding units to the backing, attachment of the lath and scratch coat to the structure, control curvature of the backing, account for differential movement, consider freeze-thaw cycling, water penetration, air leakage, and vapor diffusion. If sheathing is present, it should only be considered part of the backing if it is shown to have appropriate strength and stiffness for all applied loads, including the additional vertical loads permitted by Section 13.1.2.4, and there is an established load path from the sheathing to the studs. However, rational design provisions for adhered veneer have not been fully developed. Hagel et al (2017) presented an initial design method.

If masonry units do not comply with Section 13.3.2.1, testing would need to be performed. The testing would primarily be to determine the shear bond strength and the modulus of rupture.

The design strengths are conservative values for modern dry-set mortar. The design strengths given implicitly include a strength-reduction factor and can be directly compared to strength level loads.

The out-of-plane deflection is controlled by the backing, which is governed by other standards.

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CHAPTER 14
GLASS UNIT MASONRY

TMS 402 CODE

14.1 — General

14.1.1 *Scope*
This chapter provides requirements for empirical design of non-load-bearing glass unit masonry in exterior or interior walls.

14.1.1.1 The provisions of Part 1 and Part 2 shall apply to design of glass unit masonry, except as stated in this Chapter.

14.1.1.2 Article 1.4 of TMS 602 shall not apply to glass unit masonry.

14.1.2 *General design requirements*
Design and detail glass unit masonry to accommodate differential movement.

14.1.3 *Units*

14.1.3.1 Hollow or solid glass block units shall be standard or thin units.

14.1.3.2 The specified thickness of standard units shall be at least $3^7/_8$ in. (98.4 mm).

14.1.3.3 The specified thickness of thin units shall be $3^1/_8$ in. (79.4 mm) for hollow units or 3 in. (76.2 mm) for solid units.

14.2 — Panel size

COMMENTARY

14.1 — General

14.1.1 *Scope*
Non-load-bearing glass unit masonry is used in interior and exterior walls, partitions, window openings, and as an architectural feature. Design provisions in this Code are empirical. These provisions are cited in previous codes, are based on successful performance, and are recommended by manufacturers.

14.1.1.2 Because glass unit masonry is non-loadbearing, there is no need to verify the compressive strength of masonry.

14.2 — Panel size

The Code limitations on panel size are based on structural and performance considerations. Height limits are more restrictive than length limits based on historical requirements rather than actual field experience or engineering principles. Fire resistance rating tests of assemblies may also establish limitations on panel size. Glass block manufacturers can be contacted for technical data on the fire resistance ratings of panels and local building code should be consulted for required fire resistance ratings for glass unit masonry panels. In addition, fire resistance ratings for glass unit masonry panels may be listed with Underwriters Laboratories, Inc. at www.ul.com.

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14.2.1 *Panel area and dimensions*

The maximum area of each individual standard-unit panel shall be based on the strength level wind pressure, in accordance with Figure 14.2. The maximum dimensions between structural supports shall be limited in accordance with Table 14.2.

COMMENTARY

14.2.1 *Panel area and dimensions*

Figure CC-14.2-1 presents the results of laboratory testing conducted on glass masonry panels to determine the failure pressure under lateral loading (Pittsburgh Corning (1992); Glashaus (1992); NCMA (1992); Smolenski (1992)). Testing conducted by the National Concrete Masonry Association verified the value shown for a 12-ft x 12-ft panel in this figure (NCMA (1992)). Historically, a factor of safety of 2.7 was used to determine allowable wind pressures (Smolenski (1992)). With building codes now specifying wind pressures at strength design levels, the allowable wind pressures were increased by 1.6 to obtain strength level wind pressures. Figure 14.2 indicates the strength level wind pressure, as defined in ASCE/SEI 7, that a given area of glass unit masonry panel is permitted to resist.

There is limited historical data for thin units. For this reason, the exterior use of thin units is limited to areas where the strength level wind pressure, as defined in ASCE/SEI 7, does not exceed 32 psf (1,532 Pa).

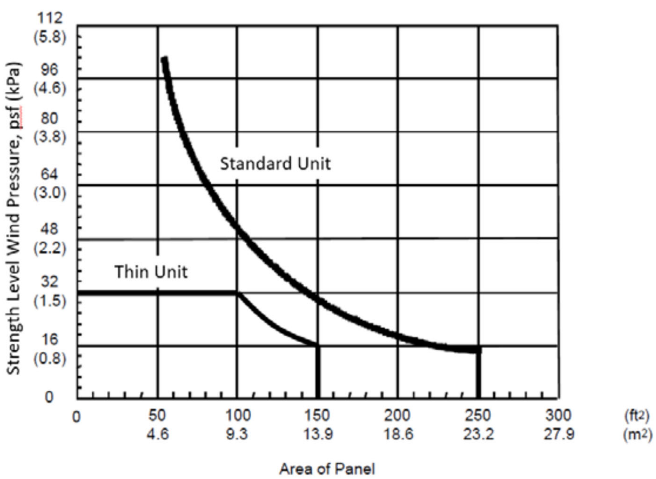

Figure 14.2 — Strength level wind pressure per ASCE/SEI 7 for glass unit masonry

Table 14.2: Glass Unit Masonry Dimension Requirements

Unit type	Maximum dimension between structural supports, ft (m)	
	Horizontal	Vertical
Standard	25 (7.62)	20 (6.10)
Thin	15 (4.57)	10 (3.05)

COMMENTARY

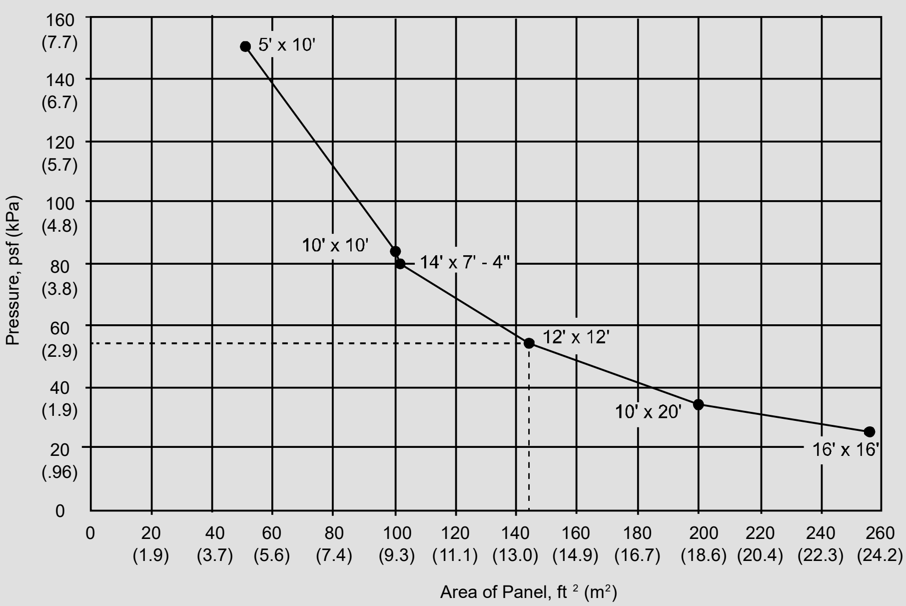

Figure CC-14.2-1 — Glass unit masonry laboratory test results

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14.2.2 *Curved panels*

The width of curved panels shall conform to the requirements of Sections 14.2.1, 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.

14.3 — Support

14.3.1 *General requirements*

Glass unit masonry panels shall be isolated so that in-plane loads are not imparted to the panel.

14.3.2 *Vertical*

14.3.2.1 Horizontally spanning members supporting glass unit masonry shall be designed so that the deflection due to allowable stress level dead plus live loads does not exceed *l*/600.

14.3.2.2 Glass unit masonry having an installed weight of 40 psf (195 kg/m^2) or less and a maximum height of 12 ft (3.7 m) shall be permitted to be supported on wood construction.

COMMENTARY

14.3 — Support

14.3.1 *General requirements*

14.3.2 *Vertical*

Support of glass unit masonry on wood has historically been permitted in model building codes. The Code requirements for expansion joints and for asphalt emulsion at the sill isolate the glass unit masonry within the wood framing. These requirements also reduce the possibility of contact of the glass units and mortar with the wood framing. The height limit of 12 ft. (3.7 m) was considered to be the maximum single story height.

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14.3.2.3 A vertical expansion joint in the glass unit masonry shall be used to isolate the glass unit masonry supported by wood construction from that supported by other types of construction.

14.3.3 *Lateral*

14.3.3.1 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 one of the following construction types, or by a combination of construction types.

(a) Panel anchors spaced not more than 16 in. (406 mm). Panel anchors shall have a minimum embedment of 12 in. (305 mm) into the mortar joint and shall have at least two fasteners per panel anchor to the support.

(b) Channel-type restraints. Glass unit masonry panels shall be recessed at least 1 in. (25.4 mm) within channels. 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 allowable stress level load of 200 lb per linear ft (2919 N/m) of panel, whichever is greater.

14.3.3.2 Glass unit masonry panels that are no more than one unit wide shall conform to the requirements of Section 14.3.3.1, except that lateral support at the top of the panel is not required.

14.3.3.3 Glass unit masonry panels that are no more than one unit high shall conform to the requirements of Section 14.3.3.1, except that lateral support at the sides of the panels is not required.

14.3.3.4 Glass unit masonry panels that are a single glass masonry unit shall conform to the requirements of Section 14.3.3.1, except that lateral support shall not be provided by panel anchors.

14.4 — 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 $^3/_8$ in. (9.5 mm) in thickness. Expansion joints shall be entirely free of mortar or other debris and shall be filled with resilient material.

14.5 — 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.

COMMENTARY

14.3.3 *Lateral*

See Figures CC-14.3-1 and CC-14.3-2 for panel anchor construction and channel-type restraint construction, respectively. Glass unit masonry panels may be laterally supported by either construction type or by a combination of construction types. The channel-type restraint construction can be made of any channel-shaped concrete, masonry, metal, or wood components so long as they provide the required lateral support.

14.5 — Base surface treatment

Current industry practice and recommendations by glass block manufacturers state that surfaces on which glass unit masonry is placed be coated with an asphalt emulsion (Pittsburgh Corning (1992); Glashau (1992)). The asphalt emulsion provides a slip plane at the panel base. This is in addition to the expansion provisions at head and jamb

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14.6 — Mortar

Type S mortar shall be specified for exterior glass unit masonry. Type S or N mortar shall be specified for interior glass unit masonry.

14.7 — Reinforcement

Glass unit masonry panels shall have horizontal joint reinforcement spaced not more than 16 in. (406 mm) on center, located in the mortar bed joint, and extending the entire length of the panel but not across expansion joints. 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 W1.7 (MW11) and have welded cross wires of size W1.7 (MW11).

COMMENTARY

locations. The asphalt emulsion also waterproofs porous panel bases.

Glass unit masonry panels subjected to structural investigation tests by the National Concrete Masonry Association (1992) to confirm the validity and use of the Glass Unit Masonry Design Wind Load Resistance chart (Figure CC-14.2-1), were constructed on bases coated with asphalt emulsion. Asphalt emulsion on glass unit masonry panel bases is needed to be consistent with these tests.

COMMENTARY

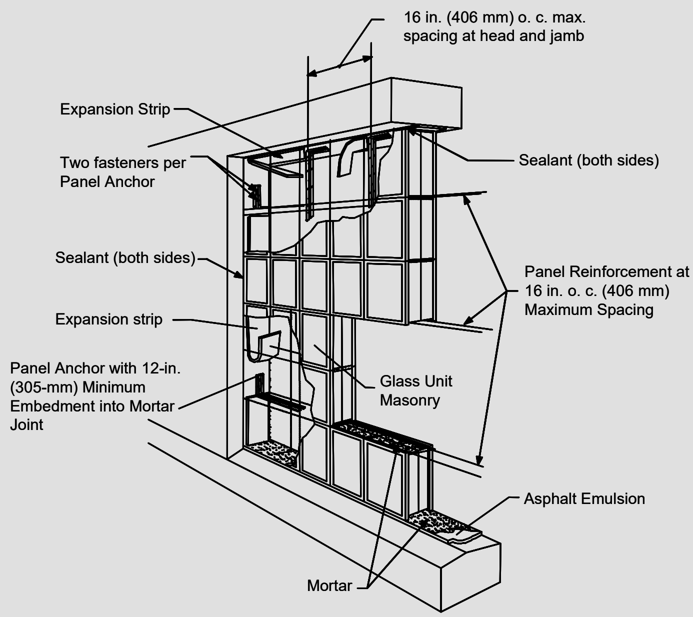

Figure CC-14.3-1 — Panel anchor construction

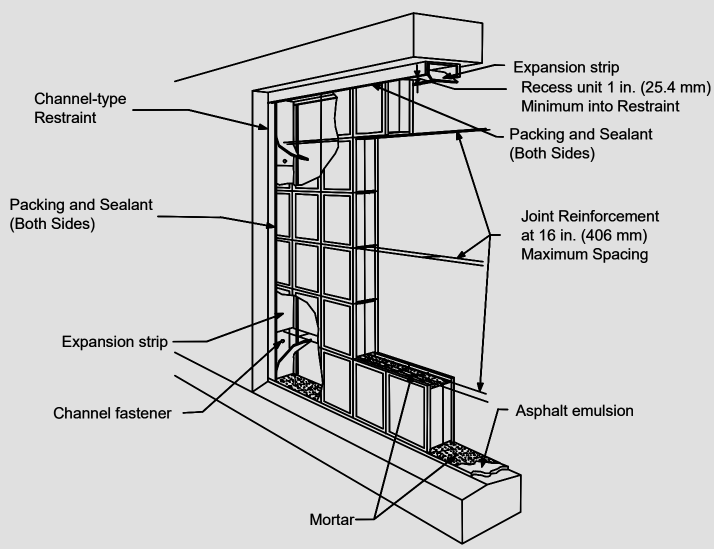

Figure CC-14.3-2 — Channel-type restraint construction

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CHAPTER 15
MASONRY PARTITION WALLS

TMS 402 CODE

15.1 — General

15.1.1 *Scope*

This chapter provides requirements for the design of masonry partition walls.

15.1.2 *Design of partition walls*

Partition walls shall be designed by one of the following:

(a) the requirements of Part 1, Part 2 and the requirements of Chapter 8, Chapter 9, Chapter 10, Chapter 11, or Chapter 14; or

(b) the prescriptive design requirements of Section 15.2 through 15.5.

15.2 — Prescriptive design of partition walls

15.2.1 *Permitted types of masonry*

15.2.1.1 Prescriptive design of partition walls shall be permitted for the following types of masonry units, when bedded in mortar that complies with TMS 602 Article 2.1 A:

(a) clay masonry units that comply with ASTM C62, C126, C216, C652, or C1405; and

(b) concrete masonry units that comply with Section 15.2.1.2 and with ASTM C55, C73, C90, C744, or C1634.

15.2.1.2 Concrete masonry units 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 25 in.^{2}/ft^2 (173,600 mm^2/m^2), or

(b) the member shall be grouted solid.

15.2.2 *General*

15.2.2.1 The provisions of Part 1 and Part 2, excluding Sections 1.2.1(c), 1.2.2, 4.1, 4.2, 4.3 and 4.34, shall apply to prescriptive design of masonry partition walls.

15.2.2.2 Article 1.4 of TMS 602 shall not apply to prescriptively designed masonry partition walls.

COMMENTARY

15.2 — Prescriptive design of partition walls

15.2.1 *Permitted types of masonry* —The prescriptive design requirements for partition walls are based on certain assumptions, including bond strength of mortar to units and use of solid units, solidly grouted units, or units with minimum face shell thickness equal to that required for hollow load-bearing concrete masonry units. Because non-load-bearing concrete masonry units and hollow clay tile are permitted by their respective ASTM standards to have thinner face shells, the prescriptive provisions of Chapter 15 do not apply to those types of units.

15.2.1.2 Concrete masonry units are required to have a normalized web area of 25 in.^{2}/ft^2 (173,600 mm^2/m^2) to allow designers to avoid checking shear stress by providing sufficient web area such that web shear stresses do not control a design. This approach is consistent with the goal of keeping the provisions of Chapter 15 more prescriptive and simplified. If the normalized web area is less than 25 in.^{2}/ft^2 (173,600 mm^2/m^2), solid grouting is required to provide additional shear area.

15.2.2 *General*

This Chapter pertains to the design of the wall member only; other members in the wall, such as lintels and pilasters, must be designed by other provisions within this Code. 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.

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15.2.3 *Limitations*

15.2.3.1 *Minimum and maximum thickness* — The minimum nominal thickness of partition walls shall be 4 in. (102 mm) and the maximum nominal thickness of partition walls shall be 12 in. (305 mm).

15.2.3.2 *Vertical loads* — The prescriptive design requirements of Chapter 15 shall not apply to the design of partition walls that support vertical compressive, allowable stress level loads of more than 200 lb/linear ft (2919 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 15 shall not apply to the design of partition walls that resist net axial tension.

15.2.3.3 *Lateral loads* — The prescriptive design requirements of Chapter 15 shall not apply to partition walls resisting allowable stress level lateral loads that exceed 50 psf (2.39 kPa).

15.2.3.4 *Nonparticipating Elements* — Partition walls designed using the prescriptive requirements of Chapter 15 shall be designed as 'nonparticipating elements' in accordance with the requirements of Section 7.3.1.

15.2.3.5 *Masonry not laid in running bond* — The prescriptive design requirements of Chapter 15 shall not apply to the design of masonry not laid in running bond in horizontally spanning walls.

15.2.3.6 *Support* — The provisions of Chapter 15 shall not apply to masonry vertically supported on wood construction.

COMMENTARY

15.2.3 *Limitations*

15.2.3.1 *Minimum and maximum thickness* — The minimum and maximum thicknesses set practical limits on walls to be designed with this simplified prescriptive method. The permitted ℓ/t or h/t values in Table 15.3.1 and Table 15.3.2 are based on analyses of partition walls ranging from 4 in. (102 mm) to 12 in. (305 mm) in nominal thickness.

15.2.3.2 *Vertical loads* — This provision allows miscellaneous light loads, such as pictures, emergency lighting, etc., to be applied to interior partition walls, while limiting the load to less than what the Code defines as a load-bearing wall, which is a wall supporting vertical loads greater than 200 lb/linear ft (2919 N/m) in addition to its own weight. The allowable stress analyses performed to establish the permitted span to thickness ratios included a 200 lb/linear ft (2919 N/m) compressive, allowable stress level load applied at the top of the wall with an eccentricity of $t/6$.

Net axial tension is not permitted in partition walls designed in accordance with this chapter.

15.2.3.3 *Lateral loads* — Combined allowable stress level out-of-plane loads acting on the partition walls must not exceed 50 psf (2.39 kPa) to use the span tables in Chapter 15. Calculated applied lateral loads should consider internal pressure due to wind and a percentage of self-weight due to seismic loading.

15.2.3.5 *Masonry not laid in running bond* — The analyses performed in establishing the permitted span to thickness ratios for the prescriptive design of partition walls were based on the allowable flexural tensile stresses for clay masonry and concrete masonry. This Code does not permit flexural tensile stress parallel to bed joints in unreinforced masonry not laid in running bond unless the masonry has a continuous grout section parallel to the span. Therefore, the prescriptive requirements of Chapter 15 limit the use of masonry that is not laid in running bond to vertically spanning walls that are solidly grouted.

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15.2.3.7 *Openings* — Openings larger than those permitted by Section 15.2.3.7.1 or with cumulative area greater than that permitted by Section 15.2.3.7.2 shall not be permitted in partition walls designed in accordance with the prescriptive provisions of Chapter 15.

15.2.3.7.1 *Maximum opening size* — Openings in partition walls shall not exceed 6 in. (152 mm) in any dimension at the face of the wall and shall not interrupt reinforcement required by Section 7.4.3.1.

15.2.3.7.2 *Maximum cumulative area of openings* — The cumulative area of openings shall not exceed 144 in.^{2} (0.093 m^2) in any 10 ft^2 (0.93 m^2) of wall surface area.

15.3 — Lateral support

15.3.1 *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 15.3.1 for walls of hollow units that are ungrouted or partially grouted and Table 15.3.2 for walls of solid units or fully grouted hollow units. Use of Table 15.3.1 and Table 15.3.2 shall be limited to partition walls that are simply supported at each end. Interpolation between tabulated values shall not be permitted. When the thickness, t, of a partition wall varies within a span, the limiting span, ℓ/t or h/t, between supports shall be governed by the smallest t.

15.3.2 *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.

COMMENTARY

15.2.3.7 *Openings* — Small openings have negligible impact on the strength or performance of the partition wall in which they are located as long as reinforcement, when required, is not displaced by the opening. Openings in excess of those permitted by Sections 15.2.3.7.1 and 15.2.3.7.2 have the potential to impact structural performance of the partition wall. Therefore, partition walls with excessive openings are required to be designed by one of the engineered design methods permitted by this Code. For the purposes of this Chapter, the term openings includes penetrations.

15.3 — Lateral support

15.3.1 *Maximum ℓ/t and h/t*

Lateral support requirements are included to limit the flexural tensile stress due to out-of-plane loads.

The permitted span to thickness ratios for prescriptively designed partition walls were established based on Allowable Stress Design using allowable stress level loads of no more than 200 lb/ft (2919 N/m) (vertical), a range of lateral out-of-plane loads, and a conservative wall self-weight. Critical sections were assumed to be at mid-span and the walls were conservatively assumed to be pinned at both supports.

Table 15.3.1 and Table 15.3.2 provide maximum ℓ/t and h/t ratios that are a function of mortar type, mortar cementitious materials, unit solidity, and extent of grouting. Second order effects of axial forces combined with progressively larger deflections were not calculated explicitly. However, the combined effects of axial and flexure loads were analyzed using Allowable Stress Design provisions. Secondary bending effects resulting from the axial loads are ignored since axial forces are limited.

The provisions of this Chapter apply to partition walls without openings. Partitions containing openings must be designed using one of the permitted engineered approaches of Part 2 of TMS 402.

15.3.2 *Cantilever walls*

The span to thickness ratios permitted for cantilevered walls are based on historical use and confirming analyses using design assumptions similar to those used to develop Table 15.3.1 and Table 15.3.2.

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15.3.3 *Support members*

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.

COMMENTARY

Table 15.3.1: Maximum *ℓ/t* or *h/t* for partition walls of ungrouted or partially grouted hollow units

Maximum combined allowable stress level out-of-plane load acting on simple span partition wall	Mortar type			
	Portland cement/lime or mortar cement		Masonry cement or air entrained portland cement/lime	
	M or S	N	M or S	N
5 psf (0.239 kPa)	26	24	22	18
10 psf (0.479 kPa)	18	16	14	12
15 psf (0.718 kPa)	15	13	12	9
20 psf (0.958 kPa)	13	11	10	8
30 psf (1.436 kPa)	10	9	8	6
40 psf (1.915 kPa)	9	8	7	5
50 psf (2.394 kPa)	8	7	6	5

Table 15.3.2: Maximum *ℓ/t* or *h/t* for partition walls of solid units or fully grouted hollow units

Maximum combined allowable stress level out-of-plane load acting on simple span partition wall	Mortar type			
	Portland cement/lime or mortar cement		Masonry cement or air entrained portland cement/lime	
	M or S	N	M or S	N
5 psf (0.239 kPa)	40	36	33	26
10 psf (0.479 kPa)	28	24	22	18
15 psf (0.718 kPa)	23	20	18	14
20 psf (0.958 kPa)	20	17	15	12
30 psf (1.436 kPa)	16	14	12	10
40 psf (1.915 kPa)	14	12	11	8
50 psf (2.394 kPa)	12	11	9	7

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COMMENTARY

15.4 — Anchorage

15.4.1 *General*
Masonry partition walls shall be anchored in accordance with Section 15.4.2 or 15.4.3.

15.4.2 *Intersecting masonry partition walls*
Masonry partition walls depending upon one another for lateral support, or upon pilasters within those walls, shall be anchored or bonded at locations where they meet or intersect by one of the following methods:

15.4.2.1 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 3 in. (76.2 mm) on the unit below.

15.4.2.2 Walls shall be anchored at their intersection at vertical intervals of not more than 16 in. (406 mm) with joint reinforcement or welded wire mesh anchors.

15.4.2.3 Other metal ties, joint reinforcement or anchors, if used, shall be spaced to provide equivalent area of anchorage to that required by Section 15.4.2.2.

15.4.3 *Anchorage to structural members*
Masonry partition walls depending upon structural masonry walls or structural frame members for lateral support shall be anchored to those members in accordance with Sections 15.4.3.1, 15.4.3.2, and 15.4.3.3.

15.4.3.1 Masonry partition walls shall be anchored so as to transfer out-of-plane lateral load acting on the partition walls to the structural members.

15.4.3.2 Masonry partition walls shall be isolated within their own plane from structural members, except as required for gravity support of the partition walls.

15.4.3.3 Isolation joints and connectors at the interfaces of masonry partition walls and structural members shall be designed to accommodate the vertical and horizontal deformations of the structural members.

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15.5 — Miscellaneous requirements

15.5.1 *Chases and recesses*
Masonry directly above chases or recesses wider than 12 in. (305 mm) shall be supported on lintels.

15.5.2 *Lap splices*
Lap splices for 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 $48d_b$.

COMMENTARY

15.5.2 *Lap splices*
The prescriptive provisions of Chapter 15 are based on partition walls that are designed as unreinforced. However, prescriptive reinforcement may still be required in partition walls to satisfy the prescriptive seismic requirements for structures assigned to Seismic Design Category C. Required lap splice lengths for reinforcement in partition walls cannot be calculated according to the allowable stress provisions of Chapter 8 or to the strength provisions of Chapter 9 because Chapter 15 does not have f'_m requirements. Therefore, a prescriptive minimum lap splice length is imposed for the prescriptive seismic reinforcement. The minimum lap splice length of 48 d_b for reinforcement is based on historical use and the use of a No. 4 (M13) bar, D 20 (MD 129) deformed wire in grout, D 2 (MD 13) deformed wire in mortar, or W1.7 (MW11) joint reinforcement.

PART 5: APPENDICES

APPENDIX A

Appendix A has been deleted in its entirety.

APPENDIX B

Appendix B has been moved to Chapter 12.

APPENDIX C
LIMIT DESIGN METHOD

TMS 402 CODE

C. *General* — The limit design method shall be permitted to be applied to a line of lateral load resistance consisting of fully grouted special reinforced masonry shear walls that are designed per the strength design provisions of Chapter 9, except that the provisions of Section 9.3.5.6 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 ASCE/SEI 7.

(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 2.3.6 of ASCE/SEI 7 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 Section 9.3.3.1.2.

(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).

COMMENTARY

C. *General* — This section provides alternative design provisions for fully grouted special reinforced masonry shear walls subjected to in-plane seismic loading. The limit design method is presented as an alternative to the requirements of Section 9.3.5.6. All other sections in Chapter 9 are applicable. The limit design method is only applicable to fully grouted special reinforced masonry shear walls because only that wall type has demonstrated sufficient ductility to achieve a proper yield mechanism. Further research is needed to extend the limit design method to other wall types, including partially grouted masonry. Limit design is considered to be particularly useful for perforated wall configurations for which a representative yield mechanism can be determined (Lepage et al (2011)).

C.1 *Yield mechanism* — This section defines the basic conditions for allowing the use of limit analysis to determine the base shear strength of a line of resistance subjected to seismic loading.

Item (a) allows the use of any of the methods of analysis permitted by ASCE/SEI 7 to determine the distribution of lateral loads; a nonlinear analysis is not required for the use of these provisions. The designer should use the seismic loading condition that produces the maximum base shear demand at the line of resistance.

Item (b) states that the location of yielding regions is assumed to occur at the interfaces between wall segments and their supporting members.

Item (c) prescribes the use of the loading condition that induces the lowest axial force due to gravity loads. For wall segments loaded with axial forces below the balanced point, this loading condition gives the lowest flexural strength and therefore leads to lower mechanism strengths. Axial loads from seismic overturning are permitted to be neglected only in the initial process of establishing the plastic capacity of the selected mechanism. Axial loads from seismic overturning are required to be considered subsequently, in determining the deformation capacity of plastic hinges.

Item (d) limits the flexural strength that is assigned to a plastic hinge so that the maximum shear that can be developed does not exceed one-half the shear strength of the wall segment. This stratagem effectively reduces the strength of the controlling yield mechanism involving wall segments vulnerable to shear failure. In addition to a reduction in strength there is a reduction in deformation capacity as indicated in C.3.2.

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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 ASCE/SEI 7.

C.3 *Mechanism deformation* — The rotational deformation demand on plastic hinges shall be determined by imposing the design displacement, ~~$1.5C_d\delta_{ne}$~~ δ_{MCE}, 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 $0.5\ \ell_w\ \varepsilon_{mu} / c$. The value of c shall be calculated for the P_u corresponding to Load Combination 6 of Section 2.3.6 of ASCE/SEI 7.

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 not greater than 1 / 400. The angular deformation capacity shall be permitted to be taken as not greater than 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 24 in. (610 mm), $\ell_w / 3$, and $h_w / 3$~~.~~;

COMMENTARY

Item (e) requires the designer to verify that the selected mechanism is the critical one. If yielding is detected away from the selected plastic hinge locations, the designer has the choice of changing the selected plastic hinge location to recognize that yielding, or of placing additional reinforcement at the section where yielding is detected.

C.2 *Mechanism strength* — Because the controlling yield mechanism is investigated using nominal strengths, an overall strength reduction factor of $\phi = 0.8$ is applied to the limiting base shear strength. For simplicity, a single value of ϕ is adopted.

C.3 *Mechanism deformation* — This section defines the ductility checks required by the limit design method. The deformation demands at locations of plastic hinges are determined by imposing the calculated inelastic roof displacement to the controlling yield mechanism. ~~The 1.5 factor in front of the term $C_d\delta_{ne}$ amplifies the displacement so that it corresponds to the displacement that would be expected for the Risk-Targeted Maximum Considered Earthquake (MCE$_R$) event. This is done~~ The displacement δ_{MCE} is used for this check to align the deformation capacity checks with the intent of ASCE/SEI 7 to have a low probability of collapse in the MCE$_R$ event. Additional commentary on δ_{MCE} is provided in the commentary for Section 9.3.5.6.2.3

C.3.1 The rotational deformation capacity is calculated assuming an ultimate curvature of ε_{mu} / c over a plastic hinge length of $0.5\ \ell_w$. The resulting expression is similar to that used in Section 9.3.5.6.2.3(a) to determine the need for special boundary elements. In the latter case, it is multiplied by wall height. The value of P_u includes earthquake effects, and may be calculated using a linearly elastic model.

C.3.2 In shear-dominated members (members whose hinge strength is assigned a value lower than their nominal flexural strength due to limitations in C.1(d)), the angular deformation capacity is limited to 1 / 400 or 1 / 200, depending on the percentage and maximum spacing of transverse and longitudinal reinforcement.

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(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 6 of Section 2.3.6 of ASCE/SEI 7 shall not exceed a compressive stress of 0.3 $f'_m A_g$ at plastic hinges in the controlling mechanism.

COMMENTARY

C.3.3 The limit of 30% of f'_m is intended to ensure that all yielding components respond below the balanced point of the *P-M* interaction diagram.

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APPENDIX D
GLASS FIBER REINFORCED POLYMER (GFRP) REINFORCED MASONRY

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D.1 — General

D.1.1 *Scope*

This appendix provides minimum requirements for design of reinforced walls and lintels consisting of grouted glass fiber reinforced polymer (GFRP) reinforcement in concrete or clay masonry.

D.1.1.1 Design of GFRP reinforced masonry shall comply with the following requirements:

(a) Part 1
(b) Chapter 4
(c) Chapter 5 excluding Sections 5.~~2~~3.1.6.2, 5.~~2~~3.2, and 5.~~3~~4
(d) Sections 6.1.2, 6.1.3.3, 6.1.3.4, 6.1.3.5, 6.1.4, 6.1.5, and 6.1.10
(e) Chapter 7 excluding Section 7.3.2
(f) Chapter 9 excluding Sections 9.1.4.3, 9.1.4.4, 9.1.9.~~3~~2, 9.3.2(e), 9.3.3.1.2, 9.3.3.2.4, 9.3.4.3, and 9.3.5.4.

For the purpose of applying the above referenced provisions, GFRP reinforcement shall be subject to the same requirements as steel reinforcing bars. Where this appendix uses notation that is defined in Chapter 2 in reference to steel reinforcement, the definition in Chapter 2 shall also apply to GFRP reinforcement.

COMMENTARY

D.1 — General

D.1.1 *Scope*

Glass fiber reinforced polymer (GFRP) reinforcement has applications in masonry near electromagnetic equipment, such as MRI rooms in hospitals and walls near high voltage cables and transformers in substations. Other applications include walls exposed to severe environments, such as in coastal construction, seawalls, and chemical plants.

The response of GFRP reinforced members to seismic loads tends to be close to linear with little energy dissipation. Therefore, when determining the seismic load using Chapter 13 (Seismic Design Requirements for Nonstructural Components) of ASCE/SEI 7, a component response modification factor, R_p, of 1.5 should be used.

GFRP reinforcing bars are generally more sensitive to elevated temperatures than steel reinforcement, which can influence the fire-resistance of GFRP reinforced masonry members. At a temperature close to the glass transition temperature, T_g, the mechanical properties of resin are reduced, resulting in reduced bond strength between the resin and the fibers. The value of T_g depends on the type of resin, but is typically in the range of 200 to 250 °F (93 to 120 °C) for resins used in GFRP bars.

D.1.1.1 Design procedures in Appendix D are strength design methods in which internal forces resulting from application of strength level loads must not exceed design strength (nominal member strength reduced by a strength-reduction factor ϕ). The design procedures are similar to those in AASHTO (2018), which are design guide specifications for GFRP reinforced concrete, and ACI 440.1R (2015) which is a guide for design and construction of FRP reinforced concrete. The provisions were verified for flexural strength of masonry walls by Galal and Sasanian (2010) and Tumialan et al. (2017), and for flexural strength of masonry beams by Galal and Enginsal (2011).

In applying the prescriptive detailing provisions such as those of Sections 4.7~~6~~, ~~5.1.1.5~~5.2.3.5 (c) and 7.4.3.1.1, GFRP reinforcing bars can be treated the same as steel reinforcing bars, as the GFRP reinforcement has a similar capacity to the steel reinforcement, since the minimum f_{fd} for straight GFRP reinforcement is 65,100 psi (449 MPa), and the GFRP reinforcement can accommodate larger strains.

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Commented [PJS6]: 20-SM-022A

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D.1.1.2 The design requirements of Appendix D shall apply only to the design of masonry walls that do not support vertical compressive, allowable stress level loads of more than 200 lb/linear ft (2919 N/m) in addition to their own weight, and lintels within such walls. The provisions of Appendix D shall not apply to the design of masonry walls that resist net axial tension.

D.1.2 *Nonparticipating elements*

Members designed in accordance with Appendix D shall be designed as 'nonparticipating elements' in accordance with Section 7.3.1.

D.1.3 *Reinforcement materials*

Reinforced masonry members shall rely upon only one reinforcement material for resisting each type of loading.

D.1.4 *Strength-reduction factors*

The value of ϕ for reinforced masonry subjected to flexure shall be taken as:

$$0.55 \quad \text{for} \quad \varepsilon_{ft} = \varepsilon_{fd}$$

$$1.55 - \frac{\varepsilon_{ft}}{\varepsilon_{fd}} \quad \text{for} \quad 0.80\varepsilon_{fd} < \varepsilon_{ft} < \varepsilon_{fd}$$

$$0.75 \quad \text{for} \quad \varepsilon_{ft} \le 0.80\varepsilon_{fd}$$

COMMENTARY

The provisions of Chapter 9 apply to the design of masonry with GFRP reinforcement except as modified by Appendix D. In particular, the P-delta provisions of Section 9.3.4.4 apply. The effective moment of inertia for determining the factored moment, M_u, including P-delta effects, can be determined from Section D.4.4.1 by replacing M_a with M_u. Given the low level of axial load permitted by this appendix, it is expected that the axial load will be considered in determining second order effects, but will be conservatively neglected in determining the section capacity; the section capacity will be checked for flexural demands only.

D.1.1.2 This provision allows miscellaneous light loads, such as pictures, emergency lighting, etc., to be applied to walls, while limiting the load to less than what the Code defines as a load-bearing wall. Net axial tension is not permitted in walls designed in accordance with this chapter.

Because most research and applications of GFRP reinforced masonry to date have focused on flexural elements, the chapter applies only to non-bearing walls, including retaining walls, and lintels, not bearing walls, shear walls, nor columns. (Provisions for bearing walls, shear walls, and columns will be developed in future editions of the Code.)

D.1.2 *Nonparticipating elements*

D.1.3 *Reinforcement materials*

Although a masonry member with both steel and GFRP reinforcement to resist the same loading condition could be analyzed using strain compatibility, research in this area is lacking. This limitation in Section D.1.3 does not prohibit using GFRP bars for resisting out-of-plane flexure and using steel joint reinforcement to resist in-plane volume change stresses, for example

D.1.4 *Strength-reduction factors*

A comparative reliability analysis of steel and GFRP-reinforced concrete beams resulted in a strength-reduction factor, ϕ, of 0.70 for tension-controlled sections and a value of 0.75 for compression-controlled sections in order to give comparable reliability indices to steel-reinforced concrete beams, or a reliability index of approximately 3.5 (Jawaheri Zadeh and Nanni, (2013)). Since the variability of masonry compressive strength is similar to the variability concrete compressive strength, these strength-reduction factors would also be applicable to masonry members. Because tension-controlled masonry members reinforced with GFRP bars do not exhibit ductile behavior, a conservative strength-reduction factor of 0.55 is specified, which is consistent with AASHTO (2018) and ACI 440.1R (2015).

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COMMENTARY

While a masonry crushing failure mode can be predicted based on calculations, the member, as constructed, may not fail accordingly. For example, if the masonry strength is higher than specified, the member can fail due to GFRP rupture. For this reason and to establish a transition between the two values of ϕ, 0.55 and 0.75, a section controlled by masonry crushing is defined as a section in which $\varepsilon_{ft} \leq 0.80\varepsilon_{fd}$ and a section controlled by GFRP rupture is defined as one in which $\varepsilon_{ft} = \varepsilon_{fd}$. The resulting relation between strength-reduction factor for flexure and tensile strain at failure in the GFRP reinforcement is illustrated in Figure CC-D.1-1.

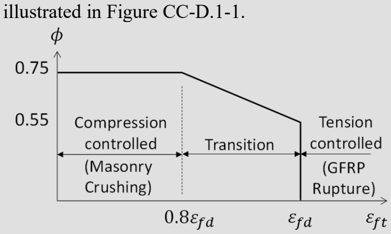

Figure CC-D.1-1 —Variation of ϕ with tensile strain at failure, ε_{ft}, in GFRP reinforcement

D.2 — Material properties

D.2.1 *Design tensile strength and strain*

The design tensile strength and design tensile strain shall be determined using Equations D-1 and D-2, where k_b shall be determined using Table D.2.1 and C_E shall be determined using Table D.2.2.

$$f_{fd} = k_b C_E f_{fu} \qquad \text{(Equation D-1)}$$

$$\varepsilon_{fd} = C_E \varepsilon_{fu} \qquad \text{(Equation D-2)}$$

Table D.2.1: GFRP Bar Bend Coefficient, k_b

GFRP Bar Condition	k_b
Straight	1.00
Bent	0.45

Table D.2.2: Environmental Reduction Factor, C_E

Exposure Condition	C_E
Masonry not exposed to earth or weather	0.80
Masonry exposed to earth or weather	0.70

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D.2 — Material properties

D.2.1 *Design tensile strength and strain*

The tensile strength reported by the manufacturer, f_{fu}, is the mean tensile strength minus three times the standard deviation. The material properties provided by the manufacturers are considered as initial properties that do not include the effects of long-term exposure to the environment. Because long-term exposure to various environments may reduce the tensile strength of GFRP bars (D'Antino et al. (2018)), the material properties used in all design equations are reduced based on type and level of environmental exposure.

The tensile strength of GFRP bars at bends is reduced due to buckling of the fibers on the inside of the bend. The value of k_b for bent bars was determined using Equation 6.2.1 from ACI 440.1R and setting the bend radius to $3d_b$, based on the minimum bend diameter permitted by TMS 602 Article 2.7 B. The coefficient k_b may be increased when larger bend radiuses are specified, using the equation:

$$k_b = \left(0.05\frac{r_b}{d_b} + 0.3\right) \leq 1.0$$

where r_b is the internal radius of bend.

For an alternative determination of the reduction in tensile strength due to bending, manufacturers of bent bars may provide test results based on the test methodologies cited in ACI 440.3R.

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COMMENTARY

ASTM D7957 requires a minimum ultimate tensile strain, ε_{fu}, of 0.011. Table CC-D.2.1 provides minimum tensile strengths, f_{fu}, required by ASTM D7957.

Table CC-D.2.1: Minimum tensile strength, f_{fu}

Bar Size	Minimum tensile strength, f_{fu}, psi (MPa)
No. 3 (M#10)	120,000 (827)
No. 4 (M#13)	108,000 (745)
No. 5 (M#16)	93,900 (647)
No. 6 (M#19)	93,000 (641)

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D.2.2 *Modulus of elasticity*
The modulus of elasticity shall be the average modulus of elasticity determined from tests.

D.2.2 *Modulus of elasticity*
ASTM D7957 requires the tensile modulus of elasticity to be greater than or equal to 6,500,000 psi (44,800 MPa).

D.3 — Reinforcement

D.3.1 *GFRP reinforcement*
GFRP reinforcement shall be deformed solid bars conforming to TMS 602 Article 2.4C.

D.3.2 *Size of GFRP reinforcement*
The maximum size of GFRP reinforcement used in masonry shall be a No. 6 (M #19).

D.3.3 *Development*
The required GFRP reinforcement shall be developed on each side of the critical section by a development length. See Section 6.1.10 for critical sections. The required development length of GFRP reinforcing bars shall be determined by Equation D-3.

$$\ell_d = \max\left\{ \frac{\dfrac{f_{fr}}{\sqrt{f'_m}} - 340}{13.6 + \dfrac{C}{d_b}} d_b, \quad 20 d_b \right\} \qquad \text{(Equation D-3)}$$

where $f_{fr} = \min\left\{ f_f, f_{fd} \right\}$.

D.3.4 *Splices*

D.3 — Reinforcement

D.3.1 *GFRP reinforcement*
GFRP bars with a smooth external surface are not permitted by this Code due to poor bond development with grout. Hollow-type GFRP bars are not permitted due to unknown performance in masonry.

D.3.2 *Size of GFRP reinforcement*
The size of GFRP reinforcement is limited because splice length testing only evaluated No. 6 (M #19) bars and smaller.

D.3.3 *Development*
The required development length is based on the stress in the GFRP bar. When the section is tension controlled, the stress will be f_{fd}. When the section is compression controlled, the stress will be f_f, which is determined from Equation D-4.

Unlike steel reinforcement, the development length of GFRP bars is based on the calculated stress in the bars. With GFRP reinforcement, all failure modes (rupture, crushing, and development/splice) are non-ductile. Lower strength-reduction factors are therefore used, making the use of the calculated stress in the bars appropriate for determining the development length.

Development lengths for GFRP bars are based on the distance to the center of the bar, while development lengths of steel reinforcement bars are based on clear distances, K. For development lengths controlled by cover, $C = K + 0.5d_b$. For development lengths controlled by bar spacing, $C = K/2 + 0.5d_b$.

The required development of dowels in concrete should be determined in accordance with ACI 440.1R.

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D.3.4 *Splices*

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The lap splice length for GFRP reinforcing bars shall not be less than 12 in. nor $1.3\ell_d$, whichever is greater.

D.3.5 *Standard hooks and bends*

D.3.5.1 Standard hooks shall be as described in TMS 602 Article 2.7 B.1.

D.3.5.2 The diameter of bend measured on the inside of reinforcing bars shall not be less than value specified in TMS 602 Article 2.7 B.2.

D.4 — Flexural members

D.4.1 *Compression reinforcement*
GFRP reinforcement in compression shall not be considered to contribute to the flexural strength of the masonry.

D.4.2 *Nominal flexural strength*

D.4.2.1 *Compression-controlled sections* — When $\varepsilon_{ft} < \varepsilon_{fd}$, failure of the member is initiated by masonry crushing. The nominal flexural strength shall be determined in accordance with the design assumptions of Section 9.3.2 as modified by Section D.4.1. The stress in the GFRP reinforcement shall be obtained from:

$$f_f = \sqrt{\frac{\left(E_f \varepsilon_{mu}\right)^2}{4} + \frac{0.64 f'_m}{\rho_f} E_f \varepsilon_{mu}} - 0.5 E_f \varepsilon_{mu}$$

(Equation D-4)

where $\rho_f = A_f / bd$.

D.4.2.2 *Tension-controlled sections* — When $\varepsilon_{ft} = \varepsilon_{fd}$, failure of the member is initiated by rupture of the GFRP reinforcing bars. The assumption of the equivalent rectangular stress block in Section 9.3.2(g) is not applicable. In lieu of a more detailed analysis, it shall be permitted to consider the masonry stress to be uniformly distributed over an equivalent compression stress block bounded by the edges of the cross-section and a straight line located parallel to the neutral axis and located at a distance $a = 0.80c_b$ from the fiber of maximum compressive strain. The value of c_b shall be determined from Equation D-5.

COMMENTARY

Although for steel reinforcement the splice length is the same as the development length for masonry structures, a splice length of 1.3 multiplied by the development length is chosen to be consistent with ACI 440.1R.

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D.3.5 *Standard hooks and bends*

D.4 — Flexural members

D.4.1 *Compression reinforcement*
The combination of the elastic modulus of GFRP reinforcement and the compression strains that are permitted in masonry result in low compression stresses in the GFRP reinforcement. Therefore, compression reinforcement is not considered to contribute to the flexural strength of the assembly.

D.4.2 *Nominal flexural strength*
The flexural resistance of a masonry member with GFRP reinforcing bars is dependent on whether the failure is governed by masonry crushing or GFRP rupture. Both failure modes are acceptable in governing the design of flexural members provided that strength and serviceability requirements are satisfied.

D.4.2.1 *Compression-controlled sections* — When member failure is initiated by masonry crushing, the stress distribution in the masonry may be approximated with the rectangular stress block approach.

D.4.2.2 *Tension-controlled sections — Tension-controlled sections* - When member failure is initiated by rupture of the GFRP reinforcement, an equivalent stress block would need to be used to approximate the stress distribution in the masonry at the particular strain level attained. A conservative simplification for determination of the masonry stress distribution is given in the Code. When using the procedure in D.4.2.2 in lieu of a more detailed analysis for tension-controlled sections, the uniform stress

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$$c_b = \left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_{fd}} \right) d \qquad \text{(Equation D-5)}$$

D.4.3 *Lintels*

The masonry stress over the equivalent compression stress block of Section 9.3.2(g) shall be 0.80$\cancel{X}$ $\chi f'_m$, where:

χ = 0.5 where the grout is not horizontally continuous in the compression zone

χ = 0.7 where the grout is continuous horizontally in the compression zone

D.4.4 *Nominal shear strength*

The nominal shear strength of GFRP reinforced masonry members shall be determined as:

$$V_n = 0.5 V_{nm} \qquad \text{(Equation D-6)}$$

where V_{nm} is determined from Section 9.3.3.1.2.1.

D.4.5 *Deflections*

D.4.5.1 — *Effective moment of inertia* - 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:

$$I_{eff} = \frac{I_{cr}}{1 - \gamma_d \left(\frac{M_{cr}}{M_a} \right)^2 \left(1 - \frac{I_{cr}}{I_n} \right)} \le I_n \qquad \text{(Equation D-7)}$$

In lieu of a more comprehensive analysis, γ_d shall be calculated as follows:

$$\gamma_d = 1.72 - 0.72 \left(\frac{M_{cr}}{M_a} \right) \qquad \text{(Equation D-8)}$$

COMMENTARY

distribution in the rectangular stress block of the masonry is assumed equal to $A_f f_{fd} / \left(0.80 c_b b\right) \le 0.8 f'_m$.

D.4.3 *Lintels*

The χ factor is used to account for a lower masonry compressive strength parallel to the bed joint than perpendicular to the bed joint. With tension-controlled sections, the compressive strength of the masonry has minimal effect on the nominal moment strength, so χ has minimal impact. For compression-controlled sections, χ has a significant impact on the strength.

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D.4.4 *Nominal shear strength*

GFRP reinforced members typically have a smaller depth to the neutral axis than steel reinforced members because of the lower of axial stiffness of the reinforcement. This results in a lower cross-section of the compression region and wider crack widths. Also, the contribution of dowel action of longitudinal GFRP reinforcement has not been determined. Because of this, the shear strength of the masonry is reduced. Equation D-6 is based on ACI 440.1R.

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Due to lack of research on the shear strength of GFRP reinforced masonry members with steel or GFRP shear reinforcement, all of the shear is required to be taken by the masonry.

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D.4.5 *Deflections*

D.4.5.1 — *Effective moment of inertia*

The section-based expression to calculate the effective moment of inertia, I_{eff}, proposed by Bischoff (2005) is modified to include the factor γ_d. This factor accounts for the variation in stiffness along the length of the member. This approach provides reasonable estimates of deflection (Bischoff et al., (2009)). The factor γ_d may be calculated using Equation D-8, which is obtained from integration of the curvature over the length of a simply supported flexural element with a uniformly distributed load. Values of γ_d for other conditions are given in Bischoff and Gross (2011).

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The cracked moment of inertia, I_{cr}, for a single layer of reinforcement and a fully grouted wall or a partially grouted wall with the neutral axis in the face shell can be obtained from the following equations:

$$I_{cr} = \frac{b(kd)^3}{3} + n_f A_f d^2 (1-k)^2$$

where k is determined from:

$$= \sqrt{\left(\rho_f n_f\right)^2 + 2\rho_f n_f} - \rho_f n_f$$

where n_f is the modular ratio of GFRP reinforcement, E_f / E_m.

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D.4.5.2 *Wall deflections* — The horizontal midheight deflection, δ_s , under allowable stress level loads shall be limited by the relation:

$$\delta_s \leq 0.01\ h \qquad \text{(Equation D-9)}$$

D.5 — Creep Rupture

The maximum sustained tensile stress in the GRFP reinforcement under allowable stress level sustained loads shall not exceed $0.3f_{fd}$. Stress in the reinforcement shall be calculated using the design assumptions of Section 8.3.2.

COMMENTARY

D.4.5.2 *Wall deflections* — The deflection limit of $0.007h$ in Section 9.3.4.5 keeps the wall in the elastic range under allowable stress level loads. A wall loaded in this range returns to its original vertical position when the lateral load is removed. With GFRP reinforcement, the wall will remain elastic at much higher levels of deflection. Thus, a higher deflection limit is allowed for GFRP reinforced masonry walls.

D.5 — Creep Rupture

This provision addresses creep rupture, a phenomenon that occurs in GFRP bars when the bar suddenly ruptures after being subjected to a constant tension over a sustained period of time. To avoid creep rupture of GFRP bars, the sustained stress level is limited to the creep rupture stress of $0.3f_{fd}$. Because this stress level will be within the elastic range of the member, the stress in the GRFP reinforcing bar can be computed through a cracked section elastic analysis.

The maximum sustained tensile stress, $f_{f,s}$, can be calculated as:

$$f_{f,s} = \frac{M_{s,s}}{A_f\left(1-k/3\right)d}$$

where $M_{s,s}$ is the moment due to dead load and other sustained loads.

Sustained loads can include fluid loads, soil pressure, and a fraction of the live load. The sustained live load is the load normally present for the intended functions of the given occupancy. Further information on sustained live loads is provided in the commentary to Chapter 4 of ASCE/SEI 7.

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EQUATION CONVERSIONS

The equations in this Code are for use with the specified inch-pound units only. The equivalent equations for use with SI units follow.

Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
Table 4.2.2	$E_m = 700 f'_m$ for clay masonry $E_m = 900 f'_m$ for concrete masonry	E_m in MPa f'_m in MPa
Table 4.2.2	$E_{AAC} = 888 (f'_{AAC})^{0.6}$	E_{AAC} in MPa f'_{AAC} in MPa
Table 4.2.2	$E_g = 500 f'_g$	E_g in MPa f'_g in MPa
(Equation 4-1)	$A_{br} = A_1\sqrt{A_2 / A_1}$	A in mm^2
(Equation 4-2)	$A_{br} = 2A_1$	A in mm^2
Table 4.4~~3~~.5	$A_{nv} = bd$	A_{nv} = mm^2 b in mm d in mm
Table 4.4~~3~~.5	$A_{nv} = t_{sp} d_v$	A_{nv} = mm^2 t_{sp} in mm d_v in mm
Table 4.4~~3~~.5	$A_{nv} = A_n$	A_{nv} = mm^2 A_n = mm^2
Table 4.4~~3~~.5	$A_{nv} = t_{sp} d$	A_{nv} = mm^2 t_{sp} in mm d in mm
Table 4.4~~3~~.5	$A_{nv} = 2t_{fs} d$	A_{nv} = mm^2 t_{fs} in mm d in mm
(Equation 5-1)	$I_{eff} = I_n \left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] \leq I_n$	I_{eff} in mm^4 I_n in mm^4 I_{cr} in mm^4 M_{cr} in N-mm M_a in N-mm
Table 5.2.2.2	(1) When $1 \leq \frac{\ell_{eff}}{d_v} < 2$, $z = 0.2(\ell_{eff} + 2d_v)$	ℓ_{eff} in mm d_v in mm z in mm
Table 5.2.2.2	(2) When $\frac{\ell_{eff}}{d_v} < 1$, $z = 0.6 l_{eff}$	ℓ_{eff} in mm d_v in mm z in mm
Table 5.2.2.2	(1) When $1 \leq \frac{\ell_{eff}}{d_v} < 3$, $z = 0.2(\ell_{eff} + 1.5d_v)$	ℓ_{eff} in mm d_v in mm z in mm
Table 5.2.2.2	(2) When $\frac{\ell_{eff}}{d_v} < 1$, $z = 0.5 l_{eff}$	ℓ_{eff} in mm d_v in mm z in mm
(Equation 6-1)	$\ell_d = 48 d_b$	ℓ_d in mm d_b in mm

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Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation 6-2)	$\ell_d = \frac{1.57 d_b^{\,2} f_y \gamma}{K\sqrt{f'_m}}$	d_b in mm f'_m in MPa $\sqrt{f'_m}$ result in MPa f_y in MPa K in mm ℓ_d in mm
(Equation 6-3)	$l_e = 13 d_b$	l_e in mm d_b in mm
(Equation 6-4)	$\xi = 1.0 - \frac{11.6 A_{sc}}{d_b^{2.5}}$ where $\frac{11.6 A_{sc}}{d_b^{2.5}} \le 1.0$	A_{sc} in mm^2 d_b in mm
6.1.10.1.5(b)	$A_v \ge 0.41\left(\frac{b_w s}{f_y}\right)$ $s \le \left(\frac{d}{8\beta_b}\right)$	A_v in mm^2 b_w in mm s in mm f_y in MPa d in mm β_b is dimensionless
(Equation 6-5)	$A_{pt} = \pi l_b^2$	A_{pt} in mm^2 l_b in mm
(Equation 6-6)	$A_{pv} = \frac{\pi l_{be}^2}{2}$	A_{pv} in mm^2 l_{be} in mm
(Equation 8-1)	$B_{ab} = 0.104 A_{pt}\sqrt{f'_m}$	A_{pt} in mm^2 B_{ab} in N f'_m in MPa $\sqrt{f'_m}$ result in MPa
(Equation 8-2)	$B_{as} = 0.5 A_b f_u$	A_b in mm^2 B_{as} in N f_u in MPa
(Equation 8-3)	$B_{ap} = 0.6 f'_m e_b d_b + 0.83\pi\left(\ell_b + e_b + d_b\right) d_b$	f'_m in MPa e_b in mm d_b in mm ℓ_b in mm B_{ap} in N
(Equation 8-4)	$B_{vb} = 0.104 A_{pv}\sqrt{f'_m}$	A_{pv} in mm^2 B_{vb} in N f'_m in MPa $\sqrt{f'_m}$ result in MPa
(Equation 8-5)	$B_{vc} = 1176\sqrt[4]{f'_m A_b}$	A_b in mm^2 B_{vc} in N f'_m in MPa $\sqrt[4]{f'_m A_b}$ result in N

Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation 8-6)	$B_{vpry} = 2.0B_{ab} = 0.208A_{pt}\sqrt{f'_m}$	A_{pt} in mm^2 B_{ab} in N B_{vpry} in N f'_m in MPa $\sqrt{f'_m}$ result in MPa
(Equation 8-7)	$B_{vs} = 0.25A_b f_u$	A_b in mm^2 B_{vs} in N f_u in MPa
(Equation 8-8)	$\left(\frac{b_a}{B_a}\right)^{5/3} + \left(\frac{b_v}{B_v}\right)^{5/3} \le 1$	b_a in N b_v in N B_a in N B_v in N
~~8.1.4~~8.1.5.2(c)	$0.108\sqrt{\text{specified unit compressive strength of header}}$	in MPa
(Equation 8-9)	$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$	F_a in MPa F_b in MPa f_a in MPa f_b in Mpa
(Equation 8-10)	$P \le \left(\frac{1}{4}\right) P_e$	P in N P_e in N
(Equation 8-11)	$F_a = \left(\frac{1}{4}\right) f'_m \left[1 - \left(\frac{h}{140r}\right)^2\right]$	F_a in MPa f'_m in MPa h in mm r in mm
(Equation 8-12)	$F_a = \left(\frac{1}{4}\right) f'_m \left(\frac{70r}{h}\right)^2$	F_a in MPa f'_m in MPa h in mm r in mm
(Equation 8-13)	$F_b = \left(\frac{1}{3}\right) f'_m$	F_b in MPa f'_m in MPa
(Equation 8-14)	$P_e = \frac{\pi^2 E_m I_n}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3$	E_m in MPa e in mm h in mm I_n in mm^4 P_e in N r in mm
(Equation 8-15)	$f_v = \frac{VQ}{Ib}$	b in mm f_v in MPa I_n in mm^4 Q in mm^3 V in N
8.2.6.2(a)	$0.125\sqrt{f'_m}$	f'_m in MPa $\sqrt{f'_m}$ result in MPa
(Equation 8-16)	$P_a = (0.30 f'_m A_n + 0.65 A_{st} F_s)\left[1 - \left(\frac{h}{140r}\right)^2\right]$	A_n in mm^2 A_{st} in mm^2 F_s in MPa f'_m in MPa h in mm P_a in N r in mm

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Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation 8-17)	$P_a = \left(0.30 f'_m A_n + 0.65 A_{st} F_s\right)\left(\frac{70r}{h}\right)^2$	A_n in mm^2 A_{st} in mm^2 F_s in MPa f'_m in MPa h in mm P_a in N r in mm
(Equation 8-18)	$\rho_{\max} = \frac{n f'_m}{2 f_y \left(n + \frac{f_y}{f'_m}\right)}$	f_y in MPa f'_m in MPa
(Equation 8-19)	$f_v = \frac{V}{A_{nv}}$	A_{nv} in mm^2 f_v in MPa V in N
(Equation 8-20)	$F_v = \left(F_{vm} + F_{vs}\right)\gamma_g$	F_v in MPa F_{vm} in MPa F_{vs} in MPa
(Equation 8-21)	$F_v \leq \left(0.249\sqrt{f'_m}\right)\gamma_g$ For $M/(Vd_v) \leq 0.25$	d_v in mm f'_m in MPa $\sqrt{f'_m}$ result in MPa F_v in MPa M in N-mm V in N
(Equation 8-22)	$F_v \leq \left(0.167\sqrt{f'_m}\right)\gamma_g$ For $M/(Vd_v) \geq 1.0$	d_v in mm f'_m in MPa $\sqrt{f'_m}$ result in MPa F_v in MPa M in N-mm V in N
(Equation 8-23)	$F_{vm} = 0.042\left[\left(4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] + 0.25\frac{P}{A_n} \geq 0$	A_n in mm^2 d_v in mm f'_m in MPa $\sqrt{f'_m}$ result in MPa F_{vm} in MPa M in N-mm P in N V in N
(Equation 8-24)	$F_{vs} = 0.5\left(\frac{A_v F_s d_v}{A_{nv} s}\right)$	A_{nv} in mm^2 A_v in mm^2 d_v in mm F_s in MPa F_{vs} in MPa s in mm
(Equation 8-25)	$F_f = \frac{\mu\left(A_{sp} F_s + P\right)}{A_{nv}} \geq 0$	A_{nv} in mm^2 A_{sp} in mm^2 F_f in MPa F_s in MPa P in N

Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation 8-26)	$F_f = \frac{0.65\left(0.75 A_{sp} F_s + P\right)}{A_{nv}} \geq 0$	A_{nv} in mm^2 A_{sp} in mm^2 F_f in MPa F_s in MPa P in N
(Equation 9-1)	$B_{anb} = 0.332 A_{pt} \sqrt{f'_m}$	A_{pt} in mm^2 B_{anb} in N f'_m in MPa $\sqrt{f'_m}$ result in MPa
(Equation 9-2)	$B_{ans} = A_b f_u$	A_b in mm^2 f_u in MPa B_{ans} in N
(Equation 9-3)	$B_{anp} = 1.5 f'_m e_b d_b + 2.07\pi\left(\ell_b + e_b + d_b\right) d_b$	f'_m in MPa e_b in mm d_b in mm ℓ_b in mm B_{anp} in N
(Equation 9-4)	$B_{vnb} = 0.332 A_{pv} \sqrt{f'_m}$	A_{pv} in mm^2 B_{vnb} in N f'_m in MPa $\sqrt{f'_m}$ result in MPa
(Equation 9-5)	$B_{vnc} = 5360 \sqrt[4]{f'_m A_b}$	A_b in mm^2 B_{vnc} in N f'_m in MPa $\sqrt[4]{f'_m A_b}$ result in N
(Equation 9-6)	$B_{vnpry} = 2.0 B_{anb} = 0.664 A_{pt} \sqrt{f'_m}$	A_{pt} in mm^2 B_{anb} in N B_{vnpry} in N f'_m in MPa $\sqrt{f'_m}$ result in MPa
(Equation 9-7)	$B_{vns} = 0.6 A_b f_u$	A_b in mm^2 f_u in MPa B_{vns} in N
(Equation 9-8)	$\left(\frac{b_{au}}{\phi B_{an}}\right)^{5/3} + \left(\frac{b_{vu}}{\phi B_{vn}}\right)^{5/3} \leq 1$	b_{au} in N b_{vu} in N B_{an} in N B_{vn} in N
9.1.7.2(c)	$0.216\sqrt{\text{specified unit compressive strength of header}}$	in MPa
(Equation 9-9)	$P_n = 0.80\left\{0.80 A_n f'_m \left[1 - \left(\frac{h}{140r}\right)^2\right]\right\}$ For $\frac{h}{r} \leq 99$	P_n in N A_n in mm^2 f'_m in MPa h in mm r in mm
(Equation 9-10)	$P_n = 0.80\left(0.80 A_n f'_m \left(\frac{70r}{h}\right)^2\right)$ For $\frac{h}{r} > 99$	P_n in N A_n in mm^2 f'_m in MPa h in mm r in mm
(Equation 9-11)	$M_u = \psi M_{u,0}$	M_u in N-mm $M_{u,0}$ in N-mm

Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation 9-12)	$\psi = \dfrac{1}{1 - \dfrac{P_u}{A_n f'_m \left(\dfrac{70r}{h}\right)^2}}$	A_n in mm^2 f'_m in MPa P_u in N h in mm r in mm
9.2.6.1(a)	$0.316 A_{nv} \sqrt{f'_m}$ in N	A_{nv} in mm^2 f'_m in MPa $\sqrt{f'_m}$ result in MPa
9.2.6.1(b)	$2.07 A_{nv}$ in N	A_{nv} in mm^2
9.2.6.2	$0.316\sqrt{f'_m} I_n b_{web} / Q$	b_{web} in mm f'_m in MPa $\sqrt{f'_m}$ result in MPa I_n in mm^4 Q in mm^3
(Equation 9-13)	$P_n = 0.80\left[0.80 f'_m \left(A_n - A_{st}\right) + f_y A_{st}\right]\left[1 - \left(\dfrac{h}{140r}\right)^2\right]$	A_n in mm^2 A_{st} in mm^2 f'_m in MPa f_y in MPa P_n in N h in mm r in mm
(Equation 9-14)	$P_n = 0.80\left[0.80 f'_m \left(A_n - A_{st}\right) + f_y A_{st}\right]\left(\dfrac{70r}{h}\right)^2$	A_n in mm^2 A_{st} in mm^2 f'_m in MPa f_y in MPa P_n in N h in mm r in mm
(Equation 9-15)	$V_n = \left(V_{nm} + V_{ns}\right)\gamma_g$	V_{nm} in N V_{ns} in N V_n in N
(Equation 9-16)	$V_n \leq \left(0.498 A_{nv} \sqrt{f'_m}\right)\gamma_g \quad \text{For} \quad \dfrac{M_u}{V_u d_v} \leq 0.25$	A_{nv} in mm^2 M_u in N-mm V_u in N d_v in mm f'_m in MPa $\sqrt{f'_m}$ result in MPa V_n in N
(Equation 9-17)	$V_n \leq \left(0.332 A_{nv} \sqrt{f'_m}\right)\gamma_g \quad \text{For} \quad \dfrac{M_u}{V_u d_v} \geq 1.0$	A_{nv} in mm^2 M_u in N-mm V_u in N f'_m in MPa $\sqrt{f'_m}$ result in MPa d_v in mm V_n in N

Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation 9-18)	$V_{nm} = 0.083\left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_{nv}\sqrt{f'_m} + 0.25P_u \geq 0$	A_{nv} in mm^2 M_u in N-mm V_u in N f'_m in MPa $\sqrt{f'_m}$ result in MPa d_v in mm P_u in N V_{nm} in N
(Equation 9-19)	$V_{ns} = 0.5\left(\frac{A_v}{s}\right) f_y\, d_v$	A_v in mm^2 f_y in MPa d_v in mm s in mm V_{ns} in N
(Equation 9-20)	$\left(\frac{P_u}{A_g}\right) \leq 0.20 f'_m$	P_u in N A_g in mm^2 f'_m in MPa
(Equation 9-21)	$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u$	h in mm w_u in N/mm P_{uf} in N e_u in mm P_u in N δ_u in mm M_u in N-mm
(Equation 9-22)	$P_u = P_{uw} + P_{uf}$	P_u in N P_{uf} in N P_{uw} in N
(Equation 9-23)	$\delta_u = \frac{5M_u h^2}{48 E_m I_n}$ For $M_u < M_{cr}$	δ_u in mm h in mm E_m in MPa I_n in mm^4 M_u in N-mm M_{cr} in N-mm
(Equation 9-24)	$\delta_u = \frac{5M_{cr}h^2}{48E_m I_n} + \frac{5\left(M_u - M_{cr}\right)h^2}{48E_m I_{cr}}$ For $M_{cr} \leq M_u \leq M_n$	δ_u in mm h in mm E_m in Mpa I_{cr} in mm^4 I_n in mm^4 M_{cr} in N-mm M_n in N-mm M_u in N-mm
(Equation 9-25)	$M_u = \psi M_{u,0}$	M_u in N-mm $M_{u,0}$ in N-mm
(Equation 9-26)	$\psi = \frac{1}{1 - \frac{P_u}{P_e}}$	P_u in N P_e in N
(Equation 9-27)	$P_e = \frac{\pi^2 E_m I_{eff}}{h^2}$	P_e in N h in mm E_m in Mpa I_{eff} in mm^4

Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation 9-28)	$I_{cr} = nA_s\left(d-c\right)^2 + \frac{nP_u}{f_y}\left(\frac{t_{sp}}{2}-c\right)^2 + \frac{bc^3}{3}$	I_{cr} in mm^4 A_s in mm^2 P_u in N t_{sp} in mm f_y in MPa d in mm c in mm b in mm
(Equation 9-29)	$c = \frac{A_s f_y + P_u}{0.64 f'_m b}$	c in mm A_s in mm^2 f_y in MPa P_u in N f'_m in MPa b in mm
(Equation 9-30)	$\delta_s \le 0.007h$	δ_s in mm h in mm
(Equation 9-31)	$V_{nf} = \mu\left(A_{sp} f_y + P_u\right) \ge 0$	V_{nf} in N A_{sp} in mm^2 f_y in MPa P_u in N
(Equation 9-32)	$V_{nf} = 0.65\left(0.75A_{sp} f_y + P_u\right) \ge 0$	V_{nf} in N A_{sp} in mm^2 f_y in MPa P_u in N
9.3.5.6.2.1(1)	$P_u \le 0.10\, A_n f'_m P_u \le 0.05\, A_n f'_m$	P_u in N A_n in mm^2 f'_m in MPa
9.3.5.6.2.1(2)	$\frac{M_u}{V_u d_v} \le 1.0$	d_v in mm M_u in N-mm V_u in N
9.3.5.6.2.1(3)	$V_u \le 0.25 A_{nv}\sqrt{f'_m}$ and $\frac{M_u}{V_u d_v} \le 3.0$	d_v in mm A_{nv} in mm^2 f'_m in MPa . $\sqrt{f'_m}$. result in MPa M_u in N-mm V_u in N
9.3.5.6.3 (a)	$c \ge \frac{\ell_w}{600\left(1.5C_d \delta_{ne} / h_w\right)}$	c in mm h_w in mm ℓ_w in mm δ_{ne} in mm
(Equation 10-1)	$f_{ps} = f_{se} + \left[\frac{0.03\left(\frac{E_{ps} d}{\ell_p}\right)\left(1-1.56\frac{A_{ps} f_{se} + P_u / \phi}{f'_m bd}\right)}{1+0.0468\left(\frac{E_{ps} A_{ps}}{f'_m b \ell_p}\right)}\right]$	f_{ps} in MPa f_{se} in MPa E_{ps} in MPa d in mm ℓ_p in mm A_{ps} in mm^2 f_{ye} in MPa P_u in N f'_m in MPa b in mm

Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation 10-2)	$a = \dfrac{f_{ps}A_{ps} + f_y A_s + P_u / \varphi}{0.80\, f'_m\, b}$	a in mm f_{ps} in MPa A_{ps} in mm^2 f_y in MPa A_s in mm^2 P_u in N f'_m in MPa b in mm
(Equation 10-3)	$M_n = \left(f_{ps}A_{ps} + f_y A_s + {P_u}/{\varphi}\right)\left(d - \dfrac{a}{2}\right)$	M_n in N-mm f_{ps} in MPa A_{ps} in mm^2 f_y in MPa A_s in mm^2 P_u in N d in mm a in mm
(Equation 10-4)	$x_t = \dfrac{f_{ps}A_{ps}\, d_{ps} + f_y A_s d + P_u t / 2\phi}{f_{ps}A_{ps} + f_y A_s + P_u / \phi}$	x_t in mm f_{ps} in MPa A_{ps} in mm^2 d_{ps} in mm f_y in MPa A_s in mm^2 d in mm P_u in N t in mm
(Equation 10-5)	$x_t = \dfrac{f_{ps}A_{ps}\, d_{ps} + f_y A_s d + P_u \ell_w / 2\phi}{f_{ps}A_{ps} + f_y A_s + P_u / \phi}$	x_t in mm f_{ps} in MPa A_{ps} in mm^2 d_{ps} in mm f_y in MPa A_s in mm^2 d in mm P_u in N ℓ_w in mm
(Equation 10-6)	$M_n = \left(f_{ps}A_{ps} + f_y A_s + \dfrac{P_u}{\varphi}\right)\left(x_t - \dfrac{a}{2}\right)$	M_n in N-mm f_{ps} in MPa A_{ps} in mm^2 f_y in MPa A_s in mm^2 P_u in N x_t in mm a in mm
(Equation 11-1)	$f_{tAAC} = 0.199\sqrt{f'_{AAC}}$	f_{tAAC} in MPa $\sqrt{f'_{AAC}}$ result in MPa
(Equation 11-2)	$f_{vd} = 0.15 f'_{AAC}$	f_{vd} in MPa f'_{AAC} in MPa
(Equation 11-3)	$P_n = 0.80\left\{0.85 A_n f'_{AAC}\left[1 - \left(\dfrac{h}{140r}\right)^2\right]\right\}$	h in mm r in mm A_n in mm^2 f'_{AAC} in MPa P_n in N

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Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation 11-4)	$P_n = 0.80\left[0.85 A_n f'_{AAC}\left(\frac{70r}{h}\right)^2\right]$	h in mm r in mm A_n in mm^2 f'_{AAC} in MPa P_n in N
(Equation 11-5)	$P_n = 0.80\left[0.85 f'_{AAC}(A_n - A_{st}) + f_y A_{st}\right]\left[1-\left(\frac{h}{140r}\right)^2\right]$	h in mm r in mm A_n in mm^2 A_{st} in mm^2 f_y in MPa f'_{AAC} in MPa P_n in N
(Equation 11-6)	$P_n = 0.80\left[0.85 f'_{AAC}(A_n - A_{st}) + f_y A_{st}\right]\left(\frac{70r}{h}\right)^2$	h in mm r in mm A_n in mm^2 A_{st} in mm^2 f_y in MPa f'_{AAC} in MPa P_n in N
(Equation 11-7)	$V_n = V_{nAAC} + V_{ns}$	V_n in N V_{nAAC} in N V_{ns} in N
(Equation 11-8)	$V_n = \mu_{AAC} P_u$	V_n in N P_u in N
(Equation 11-9)	$V_n \le 0.498 A_{nv}\sqrt{f'_{AAC}}$	V_n in N f'_{AAC} in MPa $\sqrt{f'_{AAC}}$ result in MPa A_{nv} in mm^2
(Equation 11-10)	$V_n \le 0.332 A_{nv}\sqrt{f'_{AAC}}$	V_n in N f'_{AAC} in MPa $\sqrt{f'_{AAC}}$ result in MPa A_{nv} in mm^2
(Equation 11-11a)	$V_{nAAC} = 0.0789\, l_w\, t\sqrt{f'_{AAC}}\sqrt{1+\frac{P_u}{0.199\sqrt{f'_{AAC}}\,\ell_w\, t}}$ and $1+\frac{P_u}{0.199\sqrt{f'_{AAC}}\,\ell_w\, t} \ge 0$	V_{nAAC} in N P_u in N f'_{AAC} in MPa $\sqrt{f'_{AAC}}$ result in MPa ℓ_w in mm t in mm
(Equation 11-11b)	$V_{nAAC} = 0.0548\, l_w\, t\sqrt{f'_{AAC}}\sqrt{1+\frac{P_u}{0.199\sqrt{f'_{AAC}}\,\ell_w\, t}}$ and $1+\frac{P_u}{0.199\sqrt{f'_{AAC}}\,\ell_w\, t} \ge 0$	V_{nAAC} in N P_u in N f'_{AAC} in MPa $\sqrt{f'_{AAC}}$ result in MPa ℓ_w in mm t in mm
(Equation 11-11c)	$V_{nAAC} = 0.0747\sqrt{f'_{AAC}}A_{nv} + 0.05P_u \ge 0$	V_{nAAC} in N P_u in N f'_{AAC} in MPa $\sqrt{f'_{AAC}}$ result in MPa A_{nv} in mm^2

Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation 11-12)	$V_{nAAC} = 0.17 f'_{AAC} t \dfrac{h \cdot \ell_w^{\,2}}{h^2 + (\frac{3}{4}\ell_w)^2}$	V_{nAAC} in N f'_{AAC} in MPa t in mm h in mm ℓ_w in mm
(Equation 11-13)	$V_{ns} = 0.5\left(\dfrac{A_v}{s}\right) f_y d_v$	V_{ns} in N f_y in MPa s in mm d_v in mm A_v in mm^2
(Equation 11-14)	$V_{nAAC} = 0.0664\sqrt{f'_{AAC}}\, bd$	V_{nAAC} in N f'_{AAC} in MPa $\sqrt{f'_{AAC}}$ result in MPa b in mm d in mm
(Equation 11-15)	$\dfrac{P_u}{A_g} \le 0.2 f'_{AAC}$	P_u in N f'_{AAC} in MPa A_g in mm^2
(Equation 11-16)	$M_u = \dfrac{w_u h^2}{8} + P_{uf}\dfrac{e_u}{2} + P_u \delta_u$	P_u in N P_{uf} in N h in mm e_u in mm δ_u in mm w_u in N/mm M_u in N-mm
(Equation 11-17)	$P_u = P_{uw} + P_{uf}$	P_u in N P_{uw} in N P_{uf} in N
(Equation 11-18)	$\delta_u = \dfrac{5 M_u h^2}{48 E_{AAC} I_n}$	δ_u in mm I_n in mm^4 h in mm E_{AAC} in MPa M_u in N-mm
(Equation 11-19)	$\delta_u = \dfrac{5 M_{cr} h^2}{48 E_{AAC} I_n} + \dfrac{5\left(M_u - M_{cr}\right) h^2}{48 E_{AAC} I_{cr}}$	δ_u in mm I_n in mm^4 I_{cr} in mm^4 h in mm E_{AAC} in MPa M_{cr} in N-mm M_u in N-mm
(Equation 11-20)	$M_u = \psi\, M_{u,0}$	M_u in N-mm $M_{u,0}$ in N-mm
(Equation 11-21)	$\psi = \dfrac{1}{1 - \dfrac{P_u}{P_e}}$	P_e in N P_u in N
(Equation 11-22)	$P_e = \dfrac{\pi^2 E_{AAC} I_{eff}}{h^2}$	P_e in N E_{AAC} in MPa I_{eff} in mm^4 h in mm

Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation 11-23)	$M_{cr} = S_n\left(f_{rAAC} + \frac{P}{A_n}\right)$	S_n in mm^3 A_n in mm^2 f_{rAAC} in MPa P in N M_{cr} in N-mm.
(Equation 11-24)	$I_{cr} = nA_s\left(d-c\right)^2 + \frac{nP_u}{f_y}\left(\frac{t_{sp}}{2}-c\right)^2 + \frac{bc^3}{3}$	I_{cr} in mm^4 A_s in mm^2 P_u in N t_{sp} in mm f_y in MPa d in mm c in mm b in mm
(Equation 11-25)	$c = \frac{A_s f_y + P_u}{0.57 f'_{AAC}\, b}$	c in mm A_s in mm^2 f_y in MPa P_u in N f'_{AAC} in MPa b in mm
(Equation 11-26)	$M_u \le \phi M_n$	M_u in N-mm M_n in N-mm
(Equation 11-27)	$M_n = \left(A_s f_y + P_u\right)\left(d - \frac{a}{2}\right)$	P_u in N a in mm d in mm A_s in mm^2 f_y in MPa M_n in N-mm
(Equation 11-28)	$a = \frac{\left(P_u + A_s f_y\right)}{0.85 f'_{AAC}\, b}$	a in mm P_u in N b in mm A_s in mm^2 f'_{AAC} in MPa f_y in MPa
(Equation 11-29)	$\delta_s \le 0.007\, h$	δ_s in mm h in mm
(Equation 11-30)	$V_{cr} = \frac{S_n}{h}\left(f_{rAAC} + \frac{P}{A_n}\right)$	S_n in mm^3 A_n in mm^2 h in mm f_{rAAC} in MPa P in N V_{cr} in N
11.3.6.6.2 (a)	$\cancel{c \ge} \frac{\ell_w}{600\left(C_d \delta_{ne} / h_w\right)}$ $c \ge \frac{\ell_w}{600\left(\delta_{MCE} / h_w\right)}$	c in mm h_w in mm ℓ_w in mm δ_{ne} in mm
(Equation 12-1)	$w_{\inf} = \frac{0.3}{\lambda_{strut} \cos\theta_{strut}}$	w_{inf} in. mm θ_{strut} in degrees λ_{strut} = mm^{-1}

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Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation 12-2a)	$\lambda_{strut} = \sqrt[4]{\dfrac{E_m\ t_{net\,\inf}\ \sin 2\theta_{strut}}{4\ E_{bc}\ I_{bc}\ h_{\inf}}}$	λ_{strut} = mm^{-1} E_{bc} in MPa E_m in MPa h_{inf} in mm I_{bc} in mm^4 $t_{net\,\inf}$ in mm θ_{strut} in degrees
(Equation 12-2b)	$\lambda_{strut} = \sqrt[4]{\dfrac{E_{AAC}\ t_{net\,\inf}\ \sin 2\theta_{strut}}{4\ E_{bc}\ I_{bc}\ h_{\inf}}}$	λ_{strut} = mm^{-1} E_{AAC} in MPa E_{bc} in MPa h_{inf} in mm I_{bc} in mm^4 $t_{net\,\inf}$ in mm θ_{strut} in degrees
(Equation 12-3)	$(150\,\text{mm})\ t_{net\,\inf}\ f'_m$	f'_m in MPa $t_{net\,\inf}$ in mm
(Equation 12-4)	$(150\,\text{mm})\ t_{net\,\inf}\ f'_{AAC}$	f'_{AAC} in MPa $t_{net\,\inf}$ in mm
(Equation 12-5a)	$q_{n\inf} = 729000\left(f'_m\right)^{0.75}\ t_{\inf}^2\left(\dfrac{\alpha_{arch}}{\ell_{\inf}^{2.5}} + \dfrac{\beta_{arch}}{h_{\inf}^{2.5}}\right)$	$q_{n\ \inf}$ in Pa f'_m in MPa h_{inf} in mm ℓ_{inf} in mm $t_{\inf}$ in mm α_{arch} in N$^{0.25}$ β_{arch} in N$^{0.25}$
(Equation 12-5b)	$q_{n\inf} = 729000\left(f'_{AAC}\right)^{0.75}\ t_{\inf}^2\left(\dfrac{\alpha_{arch}}{\ell_{\inf}^{2.5}} + \dfrac{\beta_{arch}}{h_{\inf}^{2.5}}\right)$	$q_{n\ \inf}$ in Pa f'_{AAC} in MPa h_{inf} in mm ℓ_{inf} in mm $t_{\inf}$ in mm α_{arch} in N$^{0.25}$ β_{arch} in N$^{0.25}$
(Equation 12-6)	$\alpha_{arch} = \dfrac{1}{h_{\inf}}(E_{bc}\ I_{bc}\ h_{\inf}^2)^{0.25} < 50$	α_{arch} in N$^{0.25}$ E_{bc} in MPa h_{inf} in mm I_{bc} in mm^4
(Equation 12-7)	$\beta_{arch} = \dfrac{1}{\ell_{\inf}}(E_{bb}\ I_{bb}\ \ell_{\inf}^2)^{0.25} < 50$	β_{arch} in N$^{0.25}$ E_{bb} in MPa ℓ_{inf} in mm I_{bb} in mm^4
(Equation C-1)	$\phi V_{lim} \geq V_{ub}$	V_{lim} in N V_{ub} in N
(Equation D-1)	$f_{fd} = k_b C_E f_{fu}$ $f_{fd} = k_b C_E f_{fu}$	f_{fd} in MPa f_{fu} in MPa
(Equation D-2)	$\varepsilon_{fd} = C_E \varepsilon_{fu}$	ε_{fd} in mm/mm ε_{fu} in mm/mm
(Equation D-3)	$l_d = \max\left\{\dfrac{12.05\dfrac{f_{fr}}{\sqrt{f'_m}} - 340}{13.6 + \dfrac{C}{d_b}} d_b, 20d_b\right\}$	C in mm d_b in mm f_{fr} in MPa $\sqrt{f'_m}$ result in MPa

Table No., Equation No., or Section No.	SI Unit Equivalent Equation	Units
(Equation D-4)	$f_f = \sqrt{\frac{\left(E_f \varepsilon_{mu}\right)^2}{4} + \frac{0.64 f_m'}{\rho_f} E_f \varepsilon_{mu}} - 0.5 E_f \varepsilon_{mu}$ $\rho_f = A_f / bd$	A_f in mm^2 b in mm d in mm E_f in MPa f_f in mm f_m' in MPa
(Equation D-5)	$c_b = \left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_{fd}}\right) d$	c_b in mm d in mm
(Equation D-6)	$V_n = 0.5 V_{nm}$	V_n in N V_{nm} in N
(Equation D-7)	$I_{eff} = \frac{I_{cr}}{1 - \gamma_d \left(\frac{M_{cr}}{M_a}\right)^2 \left(1 - \frac{I_{cr}}{I_n}\right)} \leq I_n$	I_{cr} in mm^4 I_{eff} in mm^4 I_n in mm^4 M_a in N-mm M_{cr} in N-mm
(Equation D-8)	$\gamma_d = 1.72 - 0.72 \left(\frac{M_{cr}}{M_a}\right)$	M_a in N-mm M_{cr} in N-mm
(Equation D-9)	$\delta_s \leq 0.01 h$	h in mm δ_s in mm

CONVERSION OF INCH-POUND UNITS TO SI UNITS

TO CONVERT FROM	TO	MULTIPLY BY
inches (in.)	millimeters (mm)	25.4
square inches ($in.^2$)	square millimeters (mm^2)	645.2
cubic inches ($in.^3$)	cubic millimeters (mm^3)	16,390
inches to the fourth power ($in.^4$)	millimeters to the fourth power (mm^4)	416,200
pound-force (lb)	newton (N)	4.448
pounds per linear foot (plf)	newtons per millimeter (N/mm)	0.01459
pounds per square inch (psi)	megapascal (MPa)	0.006895
pounds per square foot (psf)	kilo pascal (kPa)	0.04788
inch-pounds (in-lb)	newton-millimeters (N-mm)	113.0
$\sqrt{\text{psi}}$, result in psi	$\sqrt{\text{MPa}}$, result in MPa	0.08304

PREFIXES

POWER	PREFIX	ABBREVIATION
$1,000,000 = 10^6$	mega	M
$1,000 = 10^3$	kilo	k
$0.001 = 10^{-3}$	milli	m

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REFERENCES FOR THE CODE COMMENTARY

References, Chapter 2

BIA TN 2 (1999). “Glossary of Terms Relating to Brick Masonry,” *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

IMI (1981). *The Masonry Glossary*, International Masonry Institute.

NCMA TEK 1-4 (2004). “Glossary of Concrete Masonry Terms,” *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

References, Chapter 3

BIA TN 39 (2001). “Testing for Engineered Brick Masonry—Brick and Mortar,” *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

BIA TN 39B (1988). “Testing for Engineered Brick Masonry—Quality Control,” *Technical Notes on Brick Construciton*, Brick Industry Association, www.gobrick.com.

Chrysler, J. (~~2010~~2017). *Reinforced Concrete Masonry Construction Inspector's Handbook*, ~~7th~~ 10th Edition, Masonry Institute of America and International Code Council.

NCMA (2008). "Inspection and Testing of Concrete Masonry Construction," National Concrete Masonry Association and International Code Council.

References, Chapter 4

Argudo, J. (2003). “Evaluation and Synthesis of Experimental Data for Autoclaved Aerated Concrete,” MS Thesis, The University of Texas at Austin.

Ashour, A., and El-Dakhakhni, W. (2016). "Influence of Floor Diaphragm-Wall Coupling on the System-Level Seismic Performance of an Asymmetrical Reinforced Concrete Block Building." *Journal of Structural Engineering*, American Society of Civil Engineers, 142(10), Article 04016071.

BIA TN 3B (1993). “Brick Masonry Section Properties,” *Technical Notes on Brick Construciton*, Brick Industry Association, www.gobrick.com.

Colville, J., Miltenberger, M.A., and Wolde-Tinsae, A.M. (1993). “Hollow Concrete Masonry Modulus of Elasticity,” *6th North American Masonry Conference*, Philadelphia, PA, The Masonry Society, 1195-1208.

Copeland, R.E (1957). “Shrinkage and Temperature Stresses in Masonry,” *Journal of the American Concrete Institute,* American Concrete Institute, 53(8), 769-780.

CSA (2014). *Design of Masonry Structures*, CSA S304.1-14, Canadian Standards Association.

Galambos, T.V. and Ellingwood, B. (1986). “Serviceability Limit States: Deflection,” *Journal of Structural Engineering*, American Society of Civil Engineers, 112(1), 67-84.

Grimm, C.T. (1986). “Probabilistic Design of Expansion Joints in Brick Cladding,” *4th Canadian Masonry Symposium*, University of Fredericton, 1, 553-568.

Kalouseb, L. (1954). “Relation of Shrinkage to Moisture Content in Concrete Masonry Units,” *Paper No. 25*, Housing and Home Finance Agency, Washington, DC.

Kingsley, G.R., Shing, P. B., and Gangel, T. (2014). “Seismic Design of Special Reinforced Masonry Shear Walls, A Guide for Practicing Engineers,” *NIST GCR 14-917-31*, National Institute of Standards and Technology.

Lenczner, D., and Salahuddin, J. (1976). “Creep and Moisture Movements in Masonry Piers and Walls,” *1st Canadian Masonry Symposium*, University of Calgary, 72-86.

NCMA TEK 10-1A (2005). “Crack Control in Concrete Masonry Walls,” *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

NCMA TEK 10-~~2C~~2D (~~2010~~2019). “Control Joints for Concrete Masonry Walls – Empirical Method,” *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

NCMA TEK 10-3 (2003). “Control Joints for Concrete Masonry Walls – Alternative Engineered Method,” *e- TEK Notes*, National Concrete Masonry Association, www.ncma.org.

NCMA TEK 14-1B (2007). “Section Properties of Concrete Masonry Walls,” *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

NCMA TEK 18-2C (2014). “Sampling and Testing Concrete Masonry Units,” *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

Noland, J., and Kingsley, G. (1995). “U.S. Coordinated Program for Masonry Building Research: Technology Transfer, Research Transformed into Practice”, *Implementation of NSF Research*, Proceedings from the Conference, Arlington, Virginia, 360-371.

Code Commentary References, C-267

Commented [PJS1]:

Commented [PJS2]: 20EX-002

Commented [PJS3]: 20-EX-002

Commented [PJS4]: 20-DE-004

Plummer, H.C. (1962). *Brick and Tile Engineering*, Brick Institute of America (now Brick Industry Association).

PTI (2006). "Chapter 3- Post-Tensioning Systems," *Post-Tensioning Manual*, 6th Edition, Post-Tensioning Institute, 59-72.

RILEM (1993). *"Autoclaved Aerated Concrete Properties, Testing and Design,"* RILEM Recommended Practice, RILEM Technical Committees 78-MCA and 51-ALC.

Sarhat, S.R. and Sherwood, E.G. (2010). "Effective Shear Design of Reinforced Masonry Beams," *TMS Journal*, The Masonry Society, 28(2), 27-39.

Shing, P.B., Schuller, M.P., and Hoskere, V.S. (1990). "In-Plane Resistance of Reinforced Masonry Shear Walls," *Journal of Structural Engineering*, American Society of Civil Engineers, 116(3), 619-640.

Smith, R.G. (1973). "Moisture Expansion of Structural Ceramics – Long Term Unrestrained Expansion of Test Bricks," *Transactions of the British Ceramic Society*, British Ceramic Society, 72, 1-5.

Tanner, J.E., Varela, J.L., and Klingner, R.E. (2005). "Design and Seismic Testing of a Two-story Full-scale Autoclaved Aerated Concrete (AAC) Assemblage Specimen," *ACI Structural Journal*, American Concrete Institute, 102(1), 114-119.

Tanner, J.E., Varela, J.L., Klingner, R.E., Brightman M. J. and Cancino, U. (2005b). "Seismic Testing of Autoclaved Aerated Concrete (AAC) Shear Walls: A Comprehensive Review," *Structural Journal*, American Concrete Institute, 102(3), 374-382.

Varela, J.L., Tanner, J.E. and Klingner, R.E. (2006). "Development of Seismic Force-Reduction and Displacement Amplification Factors for AAC Structures," *Earthquake Spectra*, Earthquake Engineering Research Institute, 22(1), 267-286.

Wolde-Tinsae, A.M., Atkinson, R.H. and Hamid, A.A. (1993). "State-of-the-Art: Modulus of Elasticity," *6th North American Masonry Conference*, Philadelphia, PA, The Masonry Society, 1209-1220.

References, Chapter 5

Arora, S.K. (1988). "Performance of Masonry Walls Under Concentrated Load," *Proceedings of the British Masonry Society*, The British Masonry Society, 2, 50-55.

Bennett, R.M., McGinley, W.M., and Bryja, J. (2007). "Deflection Criteria for Masonry Beams." *Journal of ASTM International*, ASTM International, 4(1), Paper ID: JAI100442.

Branson, D.E. (1965). "Instantaneous and Time-Dependent Deflections on Simple and Continuous Reinforced Concrete Beams," *HPR Report No. 7, Part 1*, Alabama Highway Department, Bureau of Public Roads, August, 1-78.

CEB-FIP (~~1990~~2020). ~~*CEB-FIP Model Code 1990: Design Code*~~*fib Model Code for Concrete Structures 2010*. Comité Euro-International du Béton (Euro-International Committee for Concrete, CEB) and the Fédération International de la Précontrainte (International Federation for Prestressing, FIP).

CSA (2014). "Connectors for Masonry," CAN A370-14, Canadian Standards Association.

Dickey, W. and MacIntosh, A. (1971). "Results of Variation of b' or Effective Width in Flexure in Concrete Block Panels," Masonry Institute of America.

Fonseca, F.S., Mathew, S., and Bennett, R.M. (2011). "MSJC Deep Beam Requirements," *TMS Journal*, The Masonry Society, 29(1), 49-61.

Hansell, W. and Winter, G. (1959). "Lateral Stability of Reinforced Concrete Beams," *Journal of the American Concrete Institute*, American Concrete Institute, 56(5), 193-214.

He, L., and Priestley, M.J.N. (1992). "Seismic Behavior of Flanged Masonry Shear Walls - Final Report," *TCCMAR Report No. 4.1-2*, The Masonry Society.

Horton, R.T. and Tadros, M.K. (1990). "Deflection of Reinforced Masonry Members," *ACI Structural Journal*, American Concrete Institute, 87(4), 453-463.

Lee, R., Longworth, J. and Warwaruk, J. (1983). "Behavior of Restrained Masonry Beams," *3rd Canadian Masonry Symposium*, Edmonton, Alberta, 37/1-16.

NCMA TEK 12-1B (2011). "Anchors and Ties for Masonry," *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

Page, A.W., and Shrive, N.G. (1987). "Concentrated Loads on Hollow Masonry – Load Dispersion Through Bond Beams," *TMS Journal*, The Masonry Society, 6(2), T45-T51.

Park, R. and Paulay, T. (1975). *Reinforced Concrete Structures*, John Wiley & Sons.

Pfister, J.F. (1964). "Influence of Ties on the Behavior of Reinforced Concrete Columns," *Journal of the American Concrete Institute*, American Concrete Institute, 61(5), 521-537.

Revanthi, P. and Menon, D. (2006). "Estimation of Critical Buckling Moments in Slender Reinforced Concrete Beams," *ACI Structural Journal*, American Concrete Institute, 103(2), 296-303.

Commented [PJS6]: 20-EX-002

Commented [PJS5]: 20-DE-004

References, Chapter 6

ACI 318 (1983). *Building Code Requirements for Reinforced Concrete*, American Concrete Institute.

ACI 318 (2019). *Building Code Requirements for Reinforced Concrete*, American Concrete Institute.

Allen, R., Borchelt, J. G., Klingner, R. E. and Zobel, R. (2000). "Proposed Provisions for Design of Anchorage to Masonry," *TMS Journal*, The Masonry Society, 18(2), 35-59.

ASME B1.1 (2003). "Unified Inch Screw Threads, (UN and UNR Thread Form)," ASME B1.1-2003, American Society of Mechanical Engineers.

ASTM A951/A951M – 11 (2011). "Standard Specification for Steel Wire for Masonry Joint Reinforcement," ASTM International, www.astm.org.

ASTM A1064/A1064M – 13 (2013). "Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete," ASTM International, www.astm.org.

Baenziger, G., and Porter, M. L. (2018), "Joint Reinforcement as Primary Shear Reinforcement for Concrete Masonry Shear Walls," TMS Journal, 36(1), 49-64.

Brown, R.H. and Whitlock, A.R. (1983). "Strength of Anchor Bolts in Concrete Masonry," *Journal of the Structural Division*, American Society of Civil Engineers, 109(6), 1362-1374.

Dickey, W.L. (1982). "Joint Reinforcement and Masonry," *2nd North American Masonry Conference*, College Park, MD, The Masonry Society, 15-1 to 15-15.

Gallagher, E.F. (1935). "Bond Between Reinforcing Steel and Brick Masonry," *Brick and Clay Record*, 5, 86-87.

Gustafson, D. P., and Felder, A. L. (1991). "Questions and Answers on ASTM A 706 Reinforcing Bars," *Concrete International*, American Concrete Institute, 13(7), 54-57.

Hogan, M.B., Samblanet, P.J., and Thomas, R.D. (1997). "Research Evaluation of Reinforcing Bar Splices in Concrete Masonry," *11th International Brick/Block Masonry Conference*, Shanghai, China, 227-238

Mjelde, Z., McLean, D.I., Thompson, J. J. and McGinley, W. M. (2009). "Performance of Lap Splices in Concrete Masonry Shear Walls," *TMS Journal*, The Masonry Society, 27(1), 35-54.

NCMA (2009). "Effects of Confinement Reinforcement on Bar Splice Performance – Summary of Research and Design Recommendations", *Research Report MR33*, National Concrete Masonry Association, www.ncma.org.

Noland, J., and Kingsley, G. (1995). "U.S. Coordinated Program for Masonry Building Research: Technology Transfer, Research Transformed into Practice", *Implementation of NSF Research*, Proceedings from the Conference, Arlington, Virginia, 360-371.

Priestley, M.J.N., and Bridgeman, D.O. (1974). "Seismic Resistance of Brick Masonry Walls," *Bulletin,* New Zealand National Society for Earthquake Engineering (Wellington), 7(4), 167-187.

Rad, F. N, Winnen, J., M., and Mueller, W. H. (1988). "An Experimental Study on the Strength of Grouted Anchors in Masonry Walls," Report submitted to the Masonry & Ceramic Tile Institute of Oregon, Portland State University, 1988.

Richart, F.E (1949). "Bond Tests Between Steel and Mortar," Structural Clay Products Institute (now Brick Industry Association).

Rutledge, S., and DeVries, R. A., 2002, "Development of D45 Wire in Concrete," Report, School of Civil and Environmental Engineering, Oklahoma State University, Stillwater, OK, Jan., 28 pp.

Saxer, Edwin L. (1956). "Horizontal Joint Reinforcement in Concrete Masonry Construction", Report to The Dur-O-Wal Division Cedar Rapids Block Company, November 5, 1956.

Schultz, A. E. (2004). "A Reevaluation of Reinforcing Bar Splice Requirements for Masonry Structures according to the 2002 MSJC Strength Design Provisions," International Masonry Institute, Annapolis, MD, May, 2004.

Schultz, A. E. (2005). "An Evaluation of Reinforcing Bar Splice Requirements for Strength Design of Masonry Structures," Council for Masonry Research, Herndon, VA, December, 2005.

Soric, Z., and Tulin, L. (1987). "Bond in Reinforced Concrete Masonry," *4th North American Masonry Conference*, Los Angeles, CA, The Masonry Society, 47-1 - 47-17.

Treece, R.A. and Jirsa, J.O. (1989). "Bond Strength of Epoxy-Coated Reinforcing Bars," *Materials Journal*, American Concrete Institute, 86(2), 167-174.

Tubbs, J. B., Pollock, D. G. and McLean, D. I. (2000). "Testing of Anchor Bolts in Concrete Block Masonry," *TMS Journal*, The Masonry Society, 18(2), 75-88.

References, Chapter 7

Baenziger, G. P., Porter, M. L. (2011). "Joint Reinforcement for Masonry Shear Walls," *11th North American Masonry Conference*, Minneapolis, MN, The Masonry Society, Paper No. 51.

Code Commentary References, C-269

Commented [PJS7]: 21-EX-004

Baenziger, G., and Porter, M. L. (2018), "Joint Reinforcement as Primary Shear Reinforcement for Concrete Masonry Shear Walls," *TMS Journal*, The Masonry Society, 36(1), 49-64.

Brown, R. and Melander, J. (1999). "Flexural Bond Strength of Unreinforced Grouted Masonry Using PCL and MC Mortars," *Eighth North American Masonry Conference*, Austin, TX, The Masonry Society.

Gulkan, P., Mayes, R.L., and Clough, R.W. (1979). "Shaking Table Study of Single-Story Masonry Houses Volumes 1 and 2," *Report No. UCB/EERC-79/23 and 24*, Earthquake Engineering Research Center, Sept. 1979.

Hamid, A., Drysdale, R., and Heidebrecht, A. (1979). "Shear Strength of Concrete Masonry Joints," *Journal of Structural Engineering*, American Society of Civil Engineers, 105(ST7), 1227-1240.

Hassanli, R., ElGawady, M.A., and Mills, J.E. (2015). "Strength and Seismic Performance Factors of Posttensioned Masonry Walls," *Journal of Structural Engineering*, American Society of Civil Engineers, 141(11), Article 04015038.

Klingner, R.E., Shing, P.B., McGinley, W.M., McLean, D.I., Okail, H., and Jo, S. (2010). "Seismic Performance Tests of Masonry and Masonry Veneer," *Journal of ASTM International*, ASTM International, 7(3), Paper ID: JAI102740.

Minaie, E., Hamid, A., and Moon, F. (2009). "Summary of the Research on Behavior of Fully Grouted Reinforced Concrete and Clay Masonry Shear Walls," *SN2902b*, Portland Cement Association, 2009.

Porter, M. L., and Baenziger, G. P. (2007). "Joint Reinforcement as Shear Reinforcement for Masonry Shear Walls", Tenth North American Masonry Conference, St. Louis, MO, The Masonry Society, 860-871.

Schultz, A.E. (1996). "Minimum Horizontal Reinforcement Requirements for Seismic Design of Masonry Walls," *TMS Journal*, The Masonry Society, 14(1), 49-64.

Schultz, A. E., Hutchinson, R. S. (2001). "Seismic Behavior of Partially-Grouted Masonry Shear Walls: Phase 2 - Effectiveness of Bed-Joint Reinforcement," *NIST GCR 01-808*, National Institute of Standards and Technology.

Seif ElDin, H.M., and Galal, K. (2017). "Effect of Reinforcement Anchorage End Detail and Spacing on Seismic Performance of Masonry Shear Walls," Engineering Structures, 157 (2018) 268-279.

Sveinsson, B. I., McNiven, H. D., Sucuoglu, H. (1985). "Cyclic Loading Tests of Masonry Single Piers – Volume 4; Additional Tests with Height to Width Ratio of 1," UCB/EERC-85/15, Earthquake Engineering Research Center.

Tanner, J.E., Varela, J.L., Klingner, R.E. (2005). "Design and Seismic Testing of a Two-story Full-scale Autoclaved Aerated Concrete (AAC) Assemblage Specimen," *Structural Journal*, American Concrete Institute, 102(1), 114-119.

Varela, J.L., Tanner, J.E. and Klingner, R.E. (2006). "Development of Seismic Force-Reduction and Displacement Amplification Factors for AAC Structures," *Earthquake Spectra*, Earthquake Engineering Research Institute, 22(1), 267-286.

References, Chapter 8

Ahmed, M.H. (1983). "Behavior of Reinforced Brick-to-Block Walls," Ph.D. dissertation, Iowa State University.

Anand, S.C. (1985). "Shear Stresses in Composite Masonry Walls," *New Analysis Techniques for Structural Masonry*, American Society of Civil Engineers, 106-127.

Anand, S.C. and Rahman, M.A. (1985). "Temperature and Creep Stresses in Composite Masonry Walls," *Advances in Analysis of Structural Masonry*, American Society of Civil Engineers, 111-133.

Anand, S.C. and Young, D.T. (1982). "A Finite Element Model for Composite Masonry," *Journal of the Structural Division*, American Society of Civil Engineers, 108(ST12), 2637-2651.

ASTM A1064/A1064M-13 (2013). "Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete," ASTM International, www.astm.org.

BIA (1969). *Recommended Practices for Engineered Brick Masonry*, Brick Institute of America (now Brick Industry Association).

Borchelt, J.G. and Tann, J.A. (1996). "Bond Strength and Water Penetration of Low IRA Brick and Mortar," *Seventh North American Masonry Conference*, South Bend, IN, The Masonry Society, 206-216.

Brown, R.H. and Melander, J.M. (1999). "Flexural Bond Strength of Unreinforced Grouted Masonry using PCL and MC Mortars," *8th North American Masonry Conference*, Austin, TX, The Masonry Society, 694-705.

Brown, R. and Palm, B. (1982). "Flexural Strength of Brick Masonry Using the Bond Wrench," *2nd North American Masonry Conference*, College Park, MD, The Masonry Society, 1-1 - 1-15.

Colville, J. (1978). "Simplified Design of Load Bearing Masonry Walls," *5th International Symposium on*

Commented [PJS8]: 20-SL-018

Loadbearing Brickwork, Publication No. 27, British Ceramic Society, London, 2171- 2234.

Colville, J. (1979). “Stress Reduction Design Factors for Masonry Walls,” *Journal of the Structural Division*, American Society of Civil Engineers, 105(ST10), 2035-2051.

Colville, J., Matty, S.A., and Wolde-Tinsae, A.M. (1987). “Shear Capacity of Mortared Collar Joints,” *4th North American Masonry Conference*, Los Angeles, CA, The Masonry Society, 2, 60-1 - 60-15.

Colville, J. (1992). “Service Load Design Equations for Unreinforced Masonry Construction,” *TMS Journal*, The Masonry Society, 11(1), 9-20.

Davis, C.L., McLean, D.I., and Ingham, J.M. (2010). “Evaluation of Design Provisions for In-Plane Shear in Masonry Walls,” *TMS Journal*, The Masonry Society, 28(2), 41-59.

Dillon, P. B. and Fonseca, F. S. (2017). “Uncertainty in partially grouted masonry strength predictions,” *13th Canadian Masonry Symposium*, Halifax, Canada.

Drysdale, R.G. and Hamid, A.A. (1984). “Effect of Grouting on the Flexural Tensile Strength of Concrete Block Masonry,” *TMS Journal*, The Masonry Society, 3(2), T-1 - T-9.

Ellingwood, B., Galambos, T.V., MacGregor, J.G., and Cornell, C.A. (1980). “Development of a Probability Based Load Criteria for American National Standard A58,” *NBS Special Publication 577*, National Bureau of Standards.

Fabrello-Streufert, A.M., Pollock, D.G., and McLean, D.I. (2003). “Anchor Bolts in Masonry under Combined Tension and Shear Loading,” *TMS Journal*, The Masonry Society, 21(1), 13-30.

Fattal, S.G. and Cattaneo, L.E. (1976). “Structural Performance of Masonry Walls Under Compression and Flexure,” *Building Science Series No. 73*, National Bureau of Standards.

Ferguson, P. M., and Matloob, F. N. (1959). “Effect of Bar Cutoff on Bond and Shear Strength of Reinforced Concrete Beams,” *Journal of the American Concrete Institute*, American Concrete Institute, 56(1), 5-24.

Gallagher, E.F. (1935). “Bond Between Reinforcing Steel and Brick Masonry,” *Brick and Clay Record*, 5, 86-87.

Gustafson, D. P., and Felder, A. L. (1991). “Questions and Answers on ASTM A 706 Reinforcing Bars,” *Concrete International*, American Concrete Institute, 13(7), 54-57.

Hamid, A.A., Drysdale, R.G., and Heidebrecht, A.C. (1979). “Shear Strength of Concrete Masonry Joints,” *Journal of the Structural Division*, American Society of Civil Engineers, 105(ST7), 1227-1240.

Hamid, A.A. (1981). “Effect of Aspect Ratio of the Unit on the Flexural Tensile Strength of Brick Masonry,” *TMS Journal*, The Masonry Society, 1(1), T-11 - T-16.

Hamid, A.A. (1985). “Bond Characteristics of Sand-Molded Brick Masonry,” *TMS Journal*, The Masonry Society, 4(1), T-18 - T-22.

Hatzinikolas, M., Longworth, J., and Warwaruk, J. (1978). “Concrete Masonry Walls,” *Structural Engineering Report No. 70*, Department of Civil Engineering, University of Alberta, Canada, Sept. 1978.

Hedstrom, E.G., Tarhini, K.M., Thomas, R.D., Dubovoy, V.S., Klingner, R.E., and Cook, R.A. (1991). “Flexural Bond Strength of Concrete Masonry Prisms using Portland Cement and Hydrated Lime Mortars,” *TMS Journal*, The Masonry Society, 9(2), 8-23.

IIT (1963). “Development of Adjustable Wall Ties,” *ARF Project* No. B869, Illinois Institute of Technology, March 1963.

Kim, Y.S. and Bennett, R.M. (2002). “Flexural Tension in Unreinforced Masonry: Evaluation of Current Specifications.” *TMS Journal*, The Masonry Society, 20(1), 23-30.

McCarthy, J.A., Brown, R.H., and Cousins, T.E. (1985). “An Experimental Study of the Shear Strength of Collar Joints in Grouted and Slushed Composite Masonry Walls,” *3rd North American Masonry Conference*, Arlington, TX, The Masonry Society, 39-1 - 39-16.

McGinley, W.M. (2006). “Design of Anchor Bolts in Masonry,” *Progress in Structural Engineering and Materials*, John Wiley & Sons, 8(4), 155–164.

Melander, J.M. and Ghosh, S.K. (1996). “Development of Specifications for Mortar Cement,” *Masonry: Esthetics, Engineering and Economy*, STP 1246, ASTM International, www.astm.org.

Minaie, M., Mota, M., Moon, F.L., and Hamid, A.A. (2010). “In-Plane Behavior of Partially Grouted Reinforced Concrete Masonry Shear Walls,” *Journal of Structural Engineering*, American Society of Civil Engineers, 136(9), 1089-1097.

Nolph, S.M. and ElGawady M.A. (2011). “In-Plane Shear Strength of Partially Grouted Masonry Shear Walls”, *11th North American Masonry Conference*, Minneapolis, MN, The Masonry Society.

Nuss, L.K., Noland, J.L., and Chinn, J. (1978). “The Parameters Influencing Shear Strength Between Clay Masonry Units and Mortar,” *North American Masonry Conference*, Boulder, CO, The Masonry Society, 13-1 - 13-27.

Plummer, H. (1962, 1977). Brick and Tile Engineering, Second & Third editions, Brick Institute of America (formerly the Structural Clay Products Institute.)

Pook, L.L., Stylianou, M.A., and Dawe, J.L. (1986). "Experimental Investigation of the Influence of Compression on the Shear Strength of Masonry Joints," *4th Canadian Masonry Symposium*, Fredericton, New Brunswick, 1053-1062.

Porter, M.L., Wolde-Tinsae, A.M., and Ahmed, M.H. (1986). "Strength Analysis of Composite Walls," *Advances in Analysis of Structural Masonry*, American Society of Civil Engineers, 94-110.

Porter, M.L., Wolde-Tinsae, A.M., and Ahmed, M.H. (1987). "Strength Design Method for Brick Composite Walls," *4th International Masonry Conference*, London.

Ribar, J. (1982). "Water Permeance of Masonry: A Laboratory Study," *Masonry: Properties and Performance*, STP-778, ASTM International, 200-221.

Schultz, A. E (1996a). "Seismic Resistance of Partially-Grouted Masonry Shear Walls," *11th World Conference on Earthquake Engineering*, Acapulco, Mexico, Paper No. 1221.

Schultz, A. E. (1996b). "Seismic Resistance of Partially-Grouted Masonry Shear Walls," *Worldwide Advances in Structural Concrete and Masonry*, Structures Congress XIV, American Society of Civil Engineers, 211-222.

Schultz, A. E. and Hutchinson, R. S. (2001). "Seismic Behavior of Partially-Grouted Masonry Shear Walls, Phase 2: Effectiveness of Bed-Joint Reinforcement," *NIST GCR 01-808*, National Institute of Standards and Technology.

Stevens, D.J. and Anand, S.C. (1985). "Shear Stresses in Composite Masonry Walls Using a 2-D Model," *3rd North American Masonry Conference*, Arlington, TX, The Masonry Society, p. 41-1 - 41-15.

Stewart, M. G. and Lawrence, S. (2000). "Bond Strength Variability and Structural Reliability of Masonry Walls in Flexure," *12th International Brick/Block Masonry Conference*, Madrid, Spain.

UBC (1997). *Uniform Building Code*, International Conference of Building Officials.

Williams, R. and Geschwinder, L. (1982). "Shear Stress Across Collar Joints in Composite Masonry," *2nd North American Masonry Conference*, College Park, MD, The Masonry Society, Paper No. 8.

Wolde-Tinsae, A.M., Porter, M.L., and Ahmed, M.H. (1985a). "Behavior of Composite Brick Walls," *7th International Brick Masonry Conference*, Melbourne, New South Wales, 2, 877-888.

Wolde-Tinsae, A.M., Porter, M.L., and Ahmed, M.H. (1985b). "Shear Strength of Composite Brick-to-Brick Panels," *3rd North American Masonry Conference*, Arlington, TX, The Masonry Society, 40-1 - 40-13.

Woodward, K. and Rankin, F. (1984). "Influence of Vertical Compressive Stress on Shear Resistance of Concrete Block Masonry Walls," *NBSIR 84-2929*, National Bureau of Standards.

Yokel, F.Y. (1971). "Stability and Load Capacity of Members with No Tensile Strength," *Journal of the Structural Division*, American Society of Civil Engineers, 97(ST7), 1913-1926.

Yokel, F.Y., and Dikkers, R.D. (1971). "Strength of Load-Bearing Masonry Walls," *Journal of the Structural Division*, American Society of Civil Engineers, 97(ST5), 1593-1609.

Yokel, F.Y., and Dikkers, R.D. (1973). Closure to "Strength of Load-Bearing Masonry Walls," *Journal of the Structural Divison*, American Society of Civil Engineers, 99(ST5), 948-950.

References, Chapter 9

Abboud, B., Hamid, A. and Harris, H. (1993). "Flexural Behavior and Strength of Reinforced Masonry Walls Built with Masonry Cement Mortar," *TMS Journal*, The Masonry Society, 12(1), 17-24.

ACI 318 (1999). *Building Code Requirements for Reinforced Concrete*, American Concrete Institute.

Allen, R., Borchelt, J.G., Klingner, R.E. and Zobel, R. (2000). "Proposed Provisions for Design of Anchorage to Masonry," *TMS Journal*, The Masonry Society, 18(2), 35-59.

Amrhein, J.E., and Lee, D.E. (1984). "Design of Reinforced Masonry Tall Slender Walls", Western States Clay Products Association.

Anderson, D. L., and Priestly, M. J. N. (1992). "In Plane Shear Strength of Masonry Walls," *6th Canadian Masonry Symposium*, Saskatoon, Saskatchewan.

Assis, G.F. and Hamid, A.A. (1990). "Compression Behavior of Concrete Masonry Prisms Under Strain Gradient", *5th North American Masonry Conference*, Urbana, IL, The Masonry Society, 615-626.

ATC (1998). *FEMA 306 – Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Basic Procedures Manual*, Applied Technology Council.

Baenziger, G. P., Porter, M. L. (2011). "Joint Reinforcement for Masonry Shear Walls," *11th North American Masonry Conference*, Minneapolis, MN, The Masonry Society, Paper No. 51.

Brown, R.H. and Whitlock, A.R. (1983). "Strength of Anchor Bolts in Concrete Masonry," *Journal of the Structural Division*, American Society of Civil Engineers, 109(6), 1362-1374.

Brown, R.H. (1987). "Compressive Stress Distribution of Grouted Hollow Clay Masonry Under Strain Gradient", *4th North American Masonry Conference*, Los Angeles, CA, The Masonry Society, 45-1 - 45-12.

Brown, R. H., Borchelt, J. G., and Burgess, R. E. (2001). "Strength of Anchor Bolts in the Top of Clay Masonry Walls," *9th Canadian Masonry Symposium*, Fredericton, New Brunswick.

Davis, C.L., McLean, D.I., and Ingham, J.M. (2010). "Evaluation of Design Provisions for In-Plane Shear in Masonry Walls," *TMS Journal*, The Masonry Society, 28(2), 41-59.

Dillon, P. B. and Fonseca, F. S. (2017). "Uncertainty in partially grouted masonry strength predictions," 13*th Canadian Masonry Symposium*, Halifax, Canada.

Farhad, A., Hernandez, J., Scheid, G., and Klingner, R.E. (2013). "Sliding Shear Resistance of Reinforced Masonry Shear Walls." *TMS Journal*, The Masonry Society, 31(1), 45-58.

Fuchs, W., Eligenhausen, R. and Breen, J. (1995). "Concrete Capacity Design (CCD) Approach for Fastening to Concrete," *Structural Journal*, American Concrete Institute, 92(1), 73-93.

Hamid, A., Hatem, M., Harris, H. and Abboud, B. (1990)." Hysteretic Response and Ductility of Reinforced Concrete Masonry Walls under Out-of-Plane Loading," *5th North American Masonry* Conference, Urbana, IL, 397-410.

Hatzinikolos, M., Longworth, J., and Warwaruk, J. (1980). "Strength and Behavior of Anchor Bolts Embedded in Concrete Masonry," *2nd Canadian Masonry Conference*, Ottawa, Ontario, 549-563.

MacGregor, J.G., Breen, J.E., and Pfrang, E.O. (1970). "Design of Slender Concrete Columns." *Journal of the American Concrete Institute*, American Concrete Institute, 67(1), 6-28.

Malik, J.B., Mendonca, J.A., and Klingner, R.E. (1982). "Effect of Reinforcing Details on the Shear Resistance of Short Anchor Bolts under Reversed Cyclic Loading," *Journal of the American Concrete Institute*, American Concrete Institute, 79(1), 3-11.

Minaie, M. Mota, M., Moon, F.L., and Hamid, A.A. (2010). "In-Plane Behavior of Partially Grouted Reinforced Concrete Masonry Shear Walls, *Journal of Structural Engineering*, American Society of Civil Engineers, 136(9), 1089-1097.

Mirza, S.A., Lee, P.M., and Morgan, D.L. (1987). "ACI Stability Resistance Factor for RC Columns." *Journal of Structural Engineering*, American Society of Civil Engineers, 113(9), 1963-1976.

Murcia-Delso, J. and Shing, P.B. (2012). "Fragility Analysis of Reinforced Masonry Shear Walls." *Earthquake Spectra*, Earthquake Engineering Research Institute, 28(4), 1523-1547.

Noland, J., and Kingsley, G. (1995). "U.S. Coordinated Program for Masonry Building Research: Technology Transfer, Research Transformed into Practice", *Implementation of NSF Research*, Proceedings from the Conference, Arlington, Virginia, 360-371.

Nolph, S.M. and ElGawady M.A. (2011)."In-Plane Shear Strength of Partially Grouted Masonry Shear Walls", *11th North American Masonry Conference*. Minneapolis, MN, The Masonry Society.

Plummer, H. (1962, 1977). Brick and Tile Engineering, Second & Third editions, Brick Institute of America (formerly the Structural Clay Products Institute.)

Commented [PJS9]: 21-EX-004

Commented [PJS10]: 21-DE-PC171

Porter, M. L., Braun, R. A. (1998). “Horizontal Reinforcement Elongation in Masonry Mortar Joints,” *Eighth Canadian Masonry Symposium*, Jasper, Canada, 599-609.

Porter, M. L. and Braun, R. L. (1999). “An Effective Gage Length for Joint Reinforcement in Masonry”, Proceedings, *Eighth North American Masonry Conference*, Austin, TX, The Masonry Society, Paper 1A.2.

Rad, F.N., Muller, W.H. and Winnen, J.M. (1998). "An Experimental Study on the Strength of Grouted Anchors in Masonry Walls,” Report to the Masonry & Ceramic Tile Institute of Oregon, Portland State University, Portland, Oregon, October 1998.

Schultz, A.E. (1996a). “Minimum Horizontal Reinforcement Requirements for Seismic Design of Masonry Walls,” *TMS Journal*, The Masonry Society, 14(1), 49-64.

Schultz, A. E. (1996b). “Seismic Resistance of Partially-Grouted Masonry Shear Walls,” *11th World Conference on Earthquake Engineering*, Acapulco, Mexico, Paper No. 1221.

Schultz, A. E. (1996c). “Seismic Resistance of Partially-Grouted Masonry Shear Walls,” *Worldwide Advances in Structural Concrete and Masonry*, ASCE Structures Congress XIV, 211-222.

Schultz, A. E., Hutchinson, R. S. (2001). “Seismic Behavior of Partially-Grouted Masonry Shear Walls: Phase 2 - Effectiveness of Bed-Joint Reinforcement,” *NIST GCR 01-808*, National Institute of Standards and Technology.

Shing, P.B., Noland, J.L., Klamerus, E., and Spaeh, H. (1989). "Inelastic Behavior of Concrete Masonry Shear Walls." *Journal of Structural Engineering*, American Society of Civil Engineers, 115(9), 2204-2225.

Shing, P.B., Schuller, M.P., Hoskere, V.S., and Carter, E. (1990a). "Flexural and Shear Response of Reinforced Masonry Walls," *Structural Journal*, American Concrete Institute, 87(6), 646-656.

Shing, P.B., Schuller, M.P., and Hoskere, V.S. (1990b). "In-Plane Resistance of Reinforced Masonry Shear Walls," *Journal of Structural Engineering*, American Society of Civil Engineers, 116(3), 619-640.

Tubbs, J.B., Pollock, D.G., Jr., McLean, D.I. and Young, T.C. (1999). "Performance of Anchor Bolts in Concrete Block Masonry", *8th North American Masonry Conference*, Austin, TX, The Masonry Society, 866-877.

UBC (1997). *Uniform Building Code*, International Conference of Building Officials.

Voon, K.C. and Ingham, J.M. (2006). “Experimental In-Plane Shear Strength Investigation of Reinforced Concrete Masonry Walls.” *Journal of Structural Engineering*, American Society of Civil Engineers, 132(3), 400-408.

Wallace, J.W. and Orakcal, K. (2002). “ACI 318-99 Provisions for Seismic Design of Structural Walls,” *Structural Journal*, American Concrete Institute, 99(4), 499-508.

Weigel, T.A., Mohsen, J.P., Burke, A., Erdmann, K. and Schad, A. (2002). “Tensile Strength of Headed Anchor Bolts in Tops of CMU Walls,” *TMS Journal*, The Masonry Society, 20(1), 61-70.

References, Chapter 10

ACI 318 (~~2014~~2019). *Building Code Requirements for Reinforced Concrete*, American Concrete Institute.

Commented [PJS11]: 20-EX-002

ASTM A416/A416M-~~12~~18 (~~2012~~2018). “Standard Specification for ~~Steel Strand~~Low-Relaxation, ~~Uncoated~~ Seven-Wire Steel Strand for Prestressed Concrete,” ASTM International, www.astm.org.

Commented [PJS12]: 20-EX-002

ASTM A421/A421M-~~10~~21 (~~2010~~2021). “Standard Specification for ~~Uncoated~~ Stress-Relieved Steel Wire for Prestressed Concrete,” ASTM International, www.astm.org.

Commented [PJS13]: 20-EX-002

ASTM A722/A722M-~~07~~18 (~~2007~~2018). “Standard Specification for ~~Uncoated~~ High-Strength Steel Bars for Prestress~~ing~~ed Concrete,” ASTM International, www.astm.org.

Commented [PJS14]: 20-EX-002

Baqi, A. and Bhandari, N.M. (2007). “Postcracking behavior of prestressed masonry in flexure,” Journal of Structural Engineering, ASCE, 133 (4):507-518.

Baqi, A., Bhandari, N.M. and Trikha, D.N. (1999). “Experimental study of prestressed masonry flexural elements,” Journal of Structural Engineering, ASCE, 125 (3):245-254.

Bean, J.R. and Schultz A.E. (2003). “Flexural Capacity of Post-Tensioned Masonry Walls: Code Review and Recommended Procedure”, *PTI Journal*, Post-Tensioning Institute, 1(1), 28-44.

Bean Popehn, J.R (2007). “Mechanics and Behavior of Slender, Post-Tensioned Masonry Walls to Transverse Loading,” Ph.D. Dissertation, University of Minnesota.

Bean, J. R., Schultz, A. E. and Drake, C. R., (2007). “Behavior of Slender, Post-Tensioned Masonry Walls Under Transverse Loading,” *Journal of Structural Engineering*, American Society of Civil Engineers, 133(11), 1541-1550.

Bean Popehn, J. R. and Schultz, A.E. (2010). “Design Provisions for Post-Tensioned Masonry Walls Loaded Out-of-Plane”, *TMS Journal*, The Masonry Society, 28(2), 9 – 26.

Bean Popehn, J.R. and Schultz, A.E. (2011). "Finite Element Models for Slender, Post-Tensioned Masonry Walls Loaded Out-of-Plane," *Journal of Structural Engineering*, American Society of Civil Engineers, 137(12), 1489-1498.

Biggs, D.T. and Ganz, H.R. (1998). "The Codification of Prestressed Masonry in the United States", *Fifth International Masonry Conference*, London, UK, 363-366.

BSI (1985). *Code of Practice for the Use of Masonry, Part 2: Reinforced and Prestressed Masonry*, BS 5628, British Standards Institution.

Curtin, W.G., Shaw, G., and Beck, J.K. (1988). *Design of Reinforced and Prestressed Masonry*, Thomas Telford Ltd.

Ganz, H.R. (1990). "Post-Tensioned Masonry Structures", *VSL Report Series,* VSL International Ltd., Berne, Switzerland, 1990.

García, J.M., Bonett, R.L., Schultz, A.E. and Ledezma, C. (2017). "Stresses at ultimate in unbonded tendons in ungrouted post-tensioned masonry beams," Engineering Structures, 141:703-714.

García, J.M., Bonett, R.L., and Schultz, A.E., Carrillo, J. and Ledezma, C. (2019). "Flexural behavior of ungrouted post-tensioned concrete masonry beams with unbonded tendons," Journal of Construction and Building Materials, 203:210-221.

Hamilton III, H.R. and Badger, C.C.R. (2000). "Creep Losses in Post-Tensioned Concrete Masonry," *TMS Journal*, The Masonry Society, 18(1), 19-30.

Lenczner, D. (1985). "Creep and Stress Relaxation in Stack-Bonded Brick Masonry Prisms, A Pilot Study," Department of Civil Engineering, Clemson University, May 1985.

Lenczner, D. (1987). "Creep and Loss of Prestress in Stack Bonded Brick Masonry Prisms, Pilot Study - Stage II," Department of Civil Engineering, University of Illinois at Urbana-Champaign, August 1987.

NCMA TEK 14-20A (2002). "Post-tensioned Concrete Masonry Wall Design", *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

PCI, (1975). "Recommendations for Estimating Prestress Losses," *Journal of the Prestressed Concrete Institute*, Prestressed Concrete Institute, 20(4), 43-75.

Phipps, M.E. and Montague, T.I. (1976). "The Design of Prestressed Concrete Blockwork Diaphragm Walls," *Aggregate Concrete Block Association*, England.

Phipps, M.E. (1992). "The Codification of Prestressed Masonry Design," *Sixth Canadian Masonry Symposium*, Saskatoon, Saskatchewan, Canada, 561-572.

PTI (2006). Post-Tensioning Institute, "Chapter 6-Detailing and Construction Procedures for Buildings," *Post-Tensioning Manual*, 6th Edition, Post-Tensioning Institute, 113-136.

Sanders, D.H., Breen, J.E., and Duncan, R.R. III, (1987). "Strength and Behavior of Closely Spaced Post-Tensioned Monostrand Anchorages," Post-Tensioning Institute.

Schultz, A.E. and Scolforo, M.J. (1991). "An Overview of Prestressed Masonry," *TMS Journal*, The Masonry Society, 10(1), 6-21.

Schultz, A.E. and Scolforo, M.J. (1992a). "Engineering Design Provisions for Prestressed Masonry, Part 1: Masonry Stresses," *TMS Journal*, The Masonry Society, 10(2), 29-47.

Schultz, A.E., and Scolforo, M.J. (1992b). "Engineering Design Provisions for Prestressed Masonry, Part 2: Steel Stresses and Other Considerations," *TMS Journal*, The Masonry Society, 10(2), 48-64.

Schultz, A.E., Bean, J.R., and Stolarski, H. K. (2003). "Resistance of Slender Post-Tensioned Masonry Walls with Unbonded Tendons to Transverse Loading", *9th North American Masonry Conference*, Clemson, SC, The Masonry Society, 463-474.

Scolforo, M.J. and Borchelt, J.G. (1992). "Design of Reinforced and Prestressed Slender Masonry Walls," *Innovative Large Span Structures*, The Canadian Society of Civil Engineers, 709-720.

Shrive, N.G. (1988). "Effects of Time Dependent Movements in Composite and Post-Tensioned Masonry," *Masonry International*, British Masonry Society, 2(1), 1-34.

Stierwalt, D.D. and Hamilton III, H.R. (2000). "Restraint Effectiveness in Unbonded Tendons for Post-tensioned Masonry," *Structural Journal*, American Concrete Institute, 97(6), 840-848.

TMS Responds, Vol. 19 (1), (2021), "Calculating Losses for High and Low Strength Steels in Prestressed Masonry", The Masonry Society, www.masonrysociety.org.

Woodham, D.B. and Hamilton III, H.R. (2003). "Monitoring Prestress Losses in Post-Tensioned Concrete Masonry," *9th North American Masonry Conference*, Clemson, SC, The Masonry Society, 488-498.

References, Chapter 11

Argudo, J. (2003). "Evaluation and Synthesis of Experimental Data for Autoclaved Aerated Concrete," MS Thesis, The University of Texas at Austin.

ASTM C78/C78M-~~02~~21 (~~2002~~2021). "Test Method for

Code Commentary
References, C-275

Commented [PJS15]: 21-PR-002

Commented [PJS16]: 20-EX-002

Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)," ASTM International, www.astm.org.

ASTM C1693-17 (2017) "Standard Specification for Autoclaved Aerated Concrete," ASTMInternational, www.astm.org.

Cancino, U. (2003). "Behavior of Autoclaved Aerated Concrete Shear Walls with Low-Strength AAC," MS Thesis, The University of Texas at Austin.

Costa A., Penna A., Maganes G. (2011). "Seismic Performance of Autoclaved Aerated Concrete (AAC) Masonry: From Experimental Testing of the In-plane Capacity of Walls to Building Response Simulation." *The Journal of Earthquake Engineering*, Taylor & Francis, 15(1), 1-31.

Forero M.H. and Klingner R.E. (2011). "Interface Shear Transfer between Grout and AAC Masonry," *Eleventh North American Masonry Conference*, Minneapolis, MN, The Masonry Society.

Fouad, F., Dembowski, J., and Newman, D. (2002). "Material Properties and Structural Behavior of Plain and Reinforced Components," Department of Civil and Environmental Engineering, The University of Alabama at Birmingham, February 28, 2002.

Kingsley, G.R., Tulin, L. G. and Noland, J.L. (1985). "The Influence of Water Content and Unit Absorption Properties on Grout Compressive Strength and Bond Strength in Hollow Clay Unit Masonry," *3rd North American Masonry Conference*, Arlington, TX, The Masonry Society, 7-1 - 7-12.

Tanner, J.E., Varela, J.L., Klingner, R.E. (2005a). "Design and Seismic Testing of a Two-story Full-scale Autoclaved Aerated Concrete (AAC) Assemblage Specimen," *Structural Journal*, American Concrete Institute, 102(1), 114-119.

Tanner, J.E., Varela, J.L., Klingner, R.E., Brightman M. J. and Cancino, U. (2005b). "Seismic Testing of Autoclaved Aerated Concrete (AAC) Shear Walls: A Comprehensive Review," *Structural Journal*, American Concrete Institute, 102(3), 374-382.

Tanner J.E., Varela, J.L., Klingner R.E. (2011). "Validation of Material Properties and Masonry Standards Joint Committee Code Provisions for Autoclaved Aerated Concrete Masonry." *Eleventh North American Masonry Conference*, Minneapolis, MN, The Masonry Society.

Varela, J.L., Tanner, J.E. and Klingner, R.E. (2006). "Development of Seismic Force-Reduction and Displacement Amplification Factors for AAC Structures," *Earthquake Spectra*, Earthquake Engineering Research Institute, 22(1), 267-286.

Vratsanou, V. and Langer, P. (2001). "Untersuchung des Schubtragverhaltens von Wänden aus Porenbeton-Plansteinmauerwerk" (Research on Shear Behavior of Aerated Concrete Masonry Walls), *Mauerwerk*, 5(6), 210-215.

References, Chapter 12

Abrams, D. P., Angel, R., and Uzarski, J. (1993). Transverse Strength of Damaged URM Infills," *6th North American Masonry Conference*, Philadelphia, PA, The Masonry Society, 347-358.

ASCE 41 (~~2006~~2017). "~~Seismic Rehabilitation of Existing Buildings~~Seismic Evaluation and Retrofit of Existing Buildings," *ASCE 41-~~06~~17*, American Society of Civil Engineers.

Chiou, Y., Tzeng, J., and Liou, Y., "Experimental and Analytical Study of Masonry Infilled Frames." *Journal of Structural Engineering*, American Society of Civil Engineers, 125(10), 1109-1117.

Dawe, J.L, and Seah, C.K. (1989a). "Behavior of Masonry Infilled Steel Frames," *Canadian Journal of Civil Engineering*, NRC Research Press, 16, 865-876.

Dawe, J.L., and Seah, C.K. (1989b). "Out-of-Plane Resistance of Concrete Masonry Infilled Panels," *Canadian Journal of Civil Engineering*, NRC Research Press, 16, 854-864.

Flanagan, R.D., and Bennett, R.M. (1999a). "In-plane Behavior of Structural Clay Tile Infilled Frames," *Journal of Structural Engineering*, American Socieity of Civil Engineers, 125(6), 590-599.

Flanagan, R.D., and Bennett, R.M. (1999b), "Arching of Masonry Infilled Frames: Comparison of Analytical Methods," *Practice Periodical on Structural Design and Construction*, American Society of Civil Engineers, 4(3), 105-110.

Flanagan, R.D., and Bennett, R.M. (2001). "In-Plane Analysis of Masonry Infill Materials,." *Practice Periodical on Structural Design and Construction*, American Society of Civil Engineers, 6(4), 176-182.

Henderson, R. C., Porter, M.L., Jones, W.D., Burdette, E.G. (2006). "Prior Out-of-plane Damage on the In-plane Behavior of Masonry Infilled Frames," *TMS Journal*, The Masonry Society, 24(1), 71-82.

Klingner, R. E., Rubiano, N. R., Bashandy, T. and Sweeney, S. (1997). "Evaluation and Analytical Verification of Infilled Frame Test Data," *TMS Journal*, The Masonry Society, 15(2), 33-41.

Ravichandran, S. S. (2009). "Behavior and Design of AAC Infills," Ph.D. Dissertation, The University of Texas at Austin.

Ravichandran, S. S. and Klingner, R. E. (2011a). "Behavior of Steel Moment Frames with AAC Infills," *11th North American Masonry Conference*, Minneapolis, Minnesota, The Masonry Society.

Ravichandran, S. S. and Klingner, R. E. (2011b). "Seismic Design Factors for Steel Moment Frames with Masonry Infills: Part 1," *11th North American Masonry Conference*, Minneapolis, Minnesota, The Masonry Society.

Ravichandran, S. S. and Klingner, R. E. (2011c). "Seismic Design Factors for Steel Moment Frames with Masonry Infills: Part 2," *Proceedings*, *11th North American Masonry Conference*, Minneapolis, Minnesota, The Masonry Society.

Riddington, J.R. (1984). "The Influence of Initial Gaps on Infilled Frame Behavior." *Proceedings Institution of Civil Engineers*, Institution of Civil Engineers, 77, 295-310.

Tucker, C. (2007). "Predicting the In-plane Capacity of Masonry Infilled Frames," Ph.D. Dissertation, Tennessee Technological University.

References, Chapter 13

ASTM C1823–20 (2020). "Standard Test Method for Shear Bond Strength of Adhered Dimension Stone," ASTM International, www.astm.org.

ASTM E754-14 (2014), "Test Method for Pullout Resistance of Ties and Anchors Embedded in Masonry Mortar Joints" ASTM International, www.astm.org.

BIA TN 7 (2017). "Water Penetration Resistance – Design and Detailing", *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

BIA TN 18A (~~2006~~2019). "Accommodating Expansion of Brickwork", *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

BIA TN 27 (1994). "Brick Masonry Rain Screen Walls," Technical Notes on Brick Construction, Brick Industry Association, www.gobrick.com.

BIA TN 28C (2014). "Thin Brick Veneer", *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

Brown, R.H. and Arumula, J.O. (1982). "Brick Veneer with Metal Stud Backup - An Experimental and Analytical Study," *2nd North American Masonry Conference*, College Park, MD, The Masonry Society, 13-1 to 13-20.

Choi, Y-H., and LaFave, J.M (2004). "Performance of Corrugated Metal Ties for Brick Veneer Wall Systems," *Journal of Materials in Civil Engineering*, American Society of Civil Engineers, 16(3), 202-211.

Clark, C., Danforth, L., Bryja, J. (2015), "Designing Anchored Brick Veneer Above 30 Feet with a Backing of Wood Framing", *12th North American Masonry Conference*, Denver, CO, The Masonry Society.

Clark, C.B., Fried, C.L., and Bryja, J. (2018a). "History of Cold-Formed Steel Framing Used as a Backing for Brick Veneer," *Masonry 2018 Symposium: Innovations in*

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Commented [PJS18]: 20-EX-002

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Commented [PJS20]: 20-EX-002

Collaborative Research, Development and Applications, ASTM Symposium.

Clark, C.B., Fried, C.L., and Bryja, J. (2018b). "Addressing Maximum Design Deflection for Cold-Formed Steel Framing When Used as a Backing for Brick Veneer," *Masonry 2018 Symposium: Innovations in Collaborative Research, Development and Applications*, ASTM Symposium.

CSA (1984). "Connectors for Masonry," CAN3-A370-M84, Canadian Standards Association.

Dillon, P. B. and Dalrymple, G. A. (2021). "In-Field Shear Bond Strength Testing of Adhered Masonry Veneer." *14th Canadian Masonry Symposium*, Montreal, Canada.

Drysdale, R.G., and Wilson, M.J. (1989). "Behaviour of Brick Veneer/Steel Stud Tie Systems," Part 4 of the McMaster University Laboratory Test Program on Brick Veneer/Steel Stud Wall Systems, Department of Civil Engineering and Engineering Mechanics, McMaster University, Hamilton, Ontario, Canada.

FPL (2010), *Wood handbook - Wood as an engineering material*, General Technical Report FPL-GTR-190, U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Grimm, C.T. and Yura, J.A. (1989). "Shelf Angles for Masonry Veneer," *Journal of Structural Engineering*, American Society of Civil Engineers, 115(3), 509-525.

Hagel, M., Isfeld, Al., and Rizaee, S. (2017). "Initial Design Method for Thin Masonry Veneers Individually Secured by Mortar Adhesion, *13th Canadian Masonry Symposium*, Halifax, CA.

Hochwalt, J., Bennett, R., Thompson, J., Dalrymple, A., Clark, C., and Trimble, B., (2019). "Proposed Changes to the TMS 402 Anchored Veneer Provisions", *13th North American Masonry Conference*, Salt Lake City, UT.

IBC ~~2018~~(2021). "International Building Code," International Code Council, Washington D.C., ~~2018~~2021.

ICRI (2013). "Technical Guideline 310.2 – Selecting and Specifying Concrete Surface Preparation", International Concrete Repair Institute.

Kelly, T., Goodson, M., Mayes, R., and Asher, J. (1990). "Analysis of the Behavior of Anchored Brick Veneer on Metal Stud Systems Subjected to Wind and Earthquake Forces," *Fifth North American Masonry Conference*, Urbana-Champaign, IL, The Masonry Society, 1359-1370.

Klingner, R. E., Shing, P. B., McGinley, W, M., McLean, D. M., Okail, H. and Jo, S. (2010). "Seismic Performance Tests of Masonry and Masonry Veneer", *Journal of ASTM International*, ASTM International, 7(3), Paper ID:JAI102740.

Kumar (2000). "Pressure equalization of rainscreen walls: a critical review", *Building and Environment*, 35, 161-179.

McGinley, W.M., Johnson, E.N., Bennett, R.M. (2003). "Considerations in the Design of Lintels for Masonry Veneer", *9th North American Masonry Conference*, Clemson, SC, The Masonry Society.

~~MVMA~~ NCMA (~~2017~~2021). *Installation Guide and Detailing Options for Compliance with ASTM C1780*, 5th edition, 5th printing, Masonry Veneer Manufacturers Association National Concrete Masonry Association.

NCMA TEK 5-2A (2002). "Clay and Concrete Masonry Banding Details", *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

NCMA TEK 10-4 (2001). "Crack Control for Concrete Brick and Other Concrete Masonry Veneers," *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

Page, A.W., Kautto, J., and Kleeman, P.W., (1996). "A design procedure for cavity and veneer wall ties," Masonry International, 10(2), 55-62.

Porter, M.L. (1990). *Test of Masonry Connectors for Veneer Walls*, Engineering Research Institute, Iowa State University, Ames, Iowa.

Reneckis, D., and LaFave, J.M. (2010). "Out-of-Plane Seismic Performance and Detailing of Brick Veneer Walls," *Journal of Structural Engineering*, American Society of Civil Engineers, 136(7), 781-794.

Silvester, R. S., Nicastro, A. J., Antes, R. P., Niklas, W. V. (2014). "Brick Cladding Over Wood-Framed Structures – Determining Differential Movement and Overcoming Resulting Detailing Challenges," *Structural Engineer*, February.

TMS 404 (2016) *Standard for Design of Architectural Cast Stone*, The Masonry Society.

TMS 504 (2016) *Standard for the Fabrication of Architectural Cast Stone*, The Masonry Society.

TMS 604 (2016) *Standard for Installation of Architectural Cast Stone*, The Masonry Society.

Yi, J., Baird, D., McEwen, B. and Shrive, N.G. (2003). "Analysis of load in ties in masonry veneer walls," *Canadian Journal of Civil Engineering*, 30, 850-860.

References, Chapter 14

Glashaus (1992). *WECK Glass Blocks*, Glashaus Inc.

NCMA (1992). "Structural Investigation of Pittsburgh Corning Glass Block Masonry," National Concrete

Commented [PJS24]: 20-EX-002

Commented [PJS21]: 20-VG-106

Commented [PJS22]: 21-VG-014-015

Commented [PJS23]: 20-EX-002

Masonry Association Research and Development Laboratory.

Pittsburgh Corning (1992). "PC Glass Block Products," *Installation Brochure (GB-185)*, Pittsburgh Corning Corp.

Smolenski, C.P. (1992). "A Study of Mortared PCC Glass Block Panel Lateral Load Resistance (Historical Perspective and Design Implications)," Pittsburgh Corning Corporation.

References, Appendix C

Lepage A., Dill, S., Haapala M., and Sanchez, R. (2011). “Seismic Design of Reinforced Masonry Walls: Current Methods and Proposed Limit-Design Alternative,” *11th North American Masonry Conference*, Minneapolis, Minnesota, The Masonry Society.

References, Appendix D

AASHTO (2018). *LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete*, 2nd edition, American Association of State Highway and Transportation Officials, www.transportation.org.

ACI 440.1R (2015). *Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced Polymer (FRP) Bars*, ACI International.

ACI 440.3R (2012). *Guide Test Methods for Fiber-Reinforced Polymers (FRPs) for Reinforcing or Strengthening Concrete Structures*, ACI International.

ASCE/SEI 7 (~~2016~~2022). *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, American Society of Civil Engineers.

ASTM D7957/D7957M – 17 (2017). “Standard Specification for Solid Round Glass Fiber Reinforced Polymer Bars for Concrete Reinforcement,” ASTM International, www.astm.org.

Bischoff, P.H. (2005). “Reevaluation of Deflection Prediction for Concrete Beams Reinforced with Steel and Fiber Reinforced Polymer Bars.” *Journal of Structural Engineering*, American Society of Civil Engineers, 131(5), 752–767.

Bischoff, P.H., and Gross, S.P. (2011). “Equivalent Moment of Inertia Based on Integration of Curvature.” *Journal of Composites for Construction*, 15(3), 263–273.

Bischoff, P.H., Gross, S.P., and Ospina, C.E. (2009). “The Story Behind the Proposed ACI 440 Changes to Prediction of Deflections in Reinforced Concrete.” *Serviceability of Concrete Members Reinforced with Internal/External FRP Reinforcement*, ACI SP-264, American Concrete Institute, 53–76.

D’Antino, T., Pisani, M.A., and Poggi, C. (2018). “Effect of the Environment on the Performance of GFRP Reinforcing Bars.” *Composites Part B*, 141, 123–136.

Galal, K. and Enginsal, M.A. (2011). “Flexural Behavior of GFRP-Reinforced Concrete Masonry Beams”, *Journal of Composites in Construction*, ASCE, 15(1), 21-31.

Galal, K, and Sasanian, N. (2010). “Out-of-Plane Flexural Performance of GFRP-Reinforced Masonry Walls”, *Journal of Composites in Construction*, ASCE, 14(2), 162-174.

Jawaheri Zadeh, H., and Nanni, A. (2013). “Reliability Analysis of Concrete Beams Internally Reinforced with Fiber-Reinforced Polymer Bars.” *ACI Structural Journal*, American Concrete Institute, 110(6), 1023–1031.

Tumialan, G., Torres, N. Quintant, A., and Nanni, A. (2017). “Masonry Walls Reinforced with FRP Bars Subjected to Out-of-Plane Loading,” *The 13th International Symposium on Fiber-Reinforced Polymer Reinforcement for Concrete Structures*, ACI Special Publication 327, 361-374.

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Specification for Masonry Structures (TMS 602-xx)

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Specification for Masonry Structures (TMS 602-xx)

SYNOPSIS

This Specification for Masonry Structures (TMS 602-~~16~~xx) is written as a master specification and is required by Building Code Requirements for Masonry Structures (TMS 402-~~16~~xx) to control materials, labor, and construction. Thus, this Specification addresses minimum construction requirements for masonry in structures. Included are quality assurance requirements for materials; placing, bonding, and anchoring of masonry; and placement of grout and of reinforcement. This Specification may be referenced in the Project Manual. Individual project requirements may supplement the provisions of this Specification.

Keywords: AAC masonry, anchors; autoclaved aerated concrete (AAC) masonry, clay brick; clay tile; concrete block; concrete brick; construction; construction materials; curing; grout; grouting; inspection; joints; masonry; materials handling; mortars (material and placement); quality assurance and quality control; reinforcing steel; specifications; ties; tests; tolerances.

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TMS 602 SPECIFICATION

PREFACE

P1. This Preface is included for explanatory purposes only; it does not form a part of TMS 602.

P2. TMS 602 is written in the three-part section format of the Construction Specifications Institute. The language is generally imperative and terse.

P3. TMS 602 establishes minimum construction requirements for the materials and workmanship used to construct masonry structures. It is the means by which the designer conveys to the contractor the performance expectations upon which the design is based, in accordance with TMS 402.

P4. The Checklists and the Foreword to Specification Checklists are non-mandatory and do not form a part of TMS 602.

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COMMENTARY

COMMENTARY

Part 1 of the *Building Code Requirements for Masonry Structures* (TMS 402) makes the *Specification for Masonry Structures* (TMS 602) an integral part of TMS 402. TMS 602 sets minimum construction requirements regarding the materials used in and the erection of masonry structures. Specifications are written to set minimum acceptable levels of performance for the contractor. This commentary is directed to the Architect/Engineer writing the project specifications.

This Commentary explains some of the topics that the Committee considered in developing the provisions of TMS 602. Comments on specific provisions are made under the corresponding article numbers of TMS 602.

TMS 602 is a reference standard that the Architect/Engineer may cite in the contract documents for any project. It establishes the minimum construction requirements to assure compliance of the construction with the design based on TMS 402. Owners, through their representatives (Architect/Engineer), may write requirements into contract documents that are more stringent than those of TMS 602. As an example, requirements to satisfy visual aesthetics may be added in a project specification. This can be accomplished with supplemental specifications to TMS 602.

The contractor should not be required through contract documents to comply with TMS 402 or to assume responsibility regarding design requirements of TMS 402. TMS 402 is not intended to be made a part of the contract documents.

The Preface and the Foreword to Specification Checklists contain information that explains the scope of TMS 602. The Checklists are a summary of the Articles that require a decision by the Architect/Engineer preparing the contract documents. Project specifications should include the information that relates to those Checklist items that are pertinent to the project. Each project requires response to the mandatory requirements.

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TMS 602 Specification and Commentary, S-2

PART 1 — GENERAL

TMS 602 SPECIFICATION

1.1 — Summary

1.1 A. This Specification addresses requirements for materials and construction of masonry structures. SI values shown in parentheses are provided for information only and are not part of this Specification.

1.1 B. This Specification supplements the legally adopted building code and governs the construction of masonry designed in accordance with TMS 402. In areas without a legally adopted building code, this Specification defines the minimum acceptable standards of construction practice.

1.2 — Definitions

Acceptable, accepted — Acceptable to or accepted by the Architect/Engineer.

Architect/Engineer — The architect, engineer, architectural firm, engineering firm, or architectural and engineering firm, issuing drawings and specifications, or administering the work under project specifications and project drawings, or both.

Area, gross cross-sectional — The area delineated by the out-to-out specified 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 specified dimensions.

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.

COMMENTARY

1.1 — Summary

1.1 B. This Specification defines minimum requirements relative to the composition, quality, storage, handling, and placement of masonry materials and includes provisions requiring verification that the construction achieves the quality specified. The construction must conform to these requirements in order for the TMS 402 assumptions to be valid.

1.2 — Definitions

For consistent application of this Specification, it is necessary to define terms that have particular meaning in this Specification. The definitions given are for use in application of this Specification only and do not always correspond to ordinary usage. Other terms are defined in referenced documents and those definitions are applicable. If any term is defined in both this Specification and in a referenced document, the definition in this Specification applies. Referenced documents include ASTM standards. Terminology standards include ASTM C1232 Standard Terminology of Masonry and ASTM C1180 Standard Terminology of Mortar and Grout for Unit Masonry. Definitions have been coordinated between TMS 402 and this Specification.

TMS 602 SPECIFICATION

1.2 — Definitions (Continued)

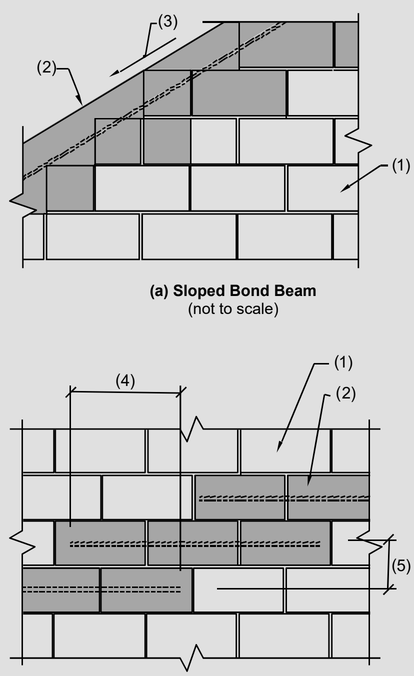
Bond beam — A horizontal, sloped, or stepped member that is fully grouted, has longitudinal reinforcement and is constructed within a masonry wall.

COMMENTARY

~~G.~~ *Bond beam* — This reinforced member is usually constructed horizontally, but may be sloped or stepped to match an adjacent roof, for example, as shown in Figure SC-1.

Notes:

(1) Masonry wall
(2) Fully grouted bond beam with reinforcement
(3) Sloped top of wall
(4) Length of noncontact lap splice
(5) Spacing between bars or deformed wires in noncontact lap splice



(a) Sloped Bond Beam
(not to scale)

(b) Stepped Bond Beam
(not to scale)

Figure SC-1— Sloped and stepped bond beams

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TMS 602 SPECIFICATION

1.2 — Definitions (Continued)

Bonded prestressing tendon — Prestressing tendon that is encapsulated by prestressing grout in a corrugated duct that is bonded to the surrounding masonry through grouting.

Cavity — The space between wythes of non-composite masonry or between a masonry veneer and its backing, which may contain insulation.

Cleanouts — Openings that are sized and spaced to allow removal of debris from the bottom of the grout space.

Collar joint — Vertical longitudinal space between wythes of composite masonry that is filled with mortar or grout.

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 this Specification.

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

Contractor — The person, firm, or corporation with whom the Owner enters into an agreement for construction of the Work.

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.

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. Thickness is given first, followed by height and then length.

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

Drainage space – A space within the cavity that allows for the drainage of water.

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

COMMENTARY

Dimension, nominal — The permitted tolerances for units are given in the appropriate materials standards. Permitted tolerances for joints and masonry construction are given in this Specification. Nominal dimensions are usually used to identify the size of a masonry unit. Nominal dimensions are normally given in whole numbers nearest to the specified dimensions.

Dimensions, specified — Specified dimensions are most often used for design calculations.

Drainage space – The drainage space may contain materials such as mortar droppings, mortar protrusions, drainage media, veneer ties, and mortar dropping collection devices provided that moisture is able to drain from the space.

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constituents during placement. (2) The hardened equivalent of such mixtures.

TMS 602 SPECIFICATION

1.2 — Definitions (Continued)

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.

Grout lift — An increment of grout height within a total grout pour. A grout pour consists of one or more grout lifts.

Grout pour — The total height of masonry to be grouted prior to erection of additional masonry. A grout pour consists of one or more grout lifts.

Inspection, continuous — Special inspection by the special inspector who is present continuously when and where the work to be inspected is being performed.

Inspection, periodic — Special inspection by the special inspector who is intermittently present where the work to be inspected has been or is being performed.

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.

Mean daily temperature — The average daily temperature of temperature extremes predicted by a local weather bureau for the next 24 hours.

Minimum daily temperature — The low temperature forecast by a local weather bureau to occur within the next 24 hours.

Minimum/maximum (not less than . . . not more than) — Minimum or maximum values given in this Specification are absolute. Do not construe that tolerances allow lowering a minimum or increasing a maximum.

Mortar — (1) A plastic mixture of cementitious materials, fine aggregates, and water, with or without admixtures, that is used to construct unit masonry assemblies. (2) The hardened equivalent of such mixtures.

Otherwise required — Specified differently in requirements supplemental to this Specification.

Owner — The public body or authority, corporation, association, partnership, or individual for whom the Work is provided.

COMMENTARY

Inspection — The Inspection Agency is required to be on the project site whenever masonry tasks requiring continuous inspection are in progress. During construction requiring periodic inspection, the Inspection Agency is only required to be on the project site intermittently, and is required to observe completed work. The frequency of periodic inspections should be defined by the Architect/Engineer as part of the quality assurance plan, and should be consistent with the complexity and size of the project.

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Partition wall — An interior wall without structural function.

TMS 602 SPECIFICATION

1.2 — Definitions (Continued)

Post-tensioning — Method of prestressing in which prestressing tendons are tensioned after the masonry has been placed.

Prestressed masonry — Masonry in which internal compressive stresses have been introduced by prestressed tendons to counteract potential tensile stresses in masonry resulting from applied loads.

Prestressing grout — A cementitious mixture used to encapsulate bonded prestressing tendons.

Prestressing tendon — Steel components such as wire, bar, strand, or a bundle of such components, used to impart prestress to masonry.

Pretensioning — Method of prestressing in which prestressing tendons are tensioned before the transfer of stress into the masonry.

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~~graphical representation that, along with the project specifications, complete the descriptive information for constructing the work~~ required by the contract documents~~.

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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.

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

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

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 project specifications or project drawings, and upon which the project design is based.

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 project specifications or project drawings, and upon which the project design is based.

COMMENTARY

~~AT.~~ *Running bond* — TMS 402 requires horizontal reinforcement in masonry that is not laid in running bond. Stack bond, which is commonly interpreted as a pattern with aligned head joints, is one bond pattern that is required to be reinforced horizontally

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TMS 602 SPECIFICATION

COMMENTARY

1.2 — Definitions (Continued)

Specified compressive strength of clay masonry or concrete masonry at the time of prestress transfer, f'_{mi} — Minimum compressive strength at the time of prestress transfer, expressed as force per unit of net cross-sectional area, required of the masonry used in construction by the project specifications or project drawings, and upon which the project design is based.

Submit, submitted — Submit, submitted to the Architect/Engineer for review.

Tendon anchorage — In post-tensioning, a device used to anchor the prestressing tendon to the masonry or concrete member; in pretensioning, a device used to anchor the prestressing tendon during hardening of masonry mortar, grout, prestressing grout, or concrete.

Tendon coupler — A device for connecting two tendon ends, thereby transferring the prestressing force from end to end.

Tendon jacking force — Temporary force exerted by a device that introduces tension into prestressing tendons.

Unbonded prestressing tendon — Prestressing tendon that is not bonded to masonry.

Veneer, adhered — Masonry veneer secured to and supported by the backing through direct bond to a masonry or concrete backing; or bond to either a scratch coat and lath or a cement backer unit that is fastened to a masonry, concrete, or light frame backing.

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

Wall — A member, usually vertical, used to enclose or separate spaces or uses.

Wall, load-bearing — A wall supporting vertical loads greater than 200 lb per linear foot (2919 N/m) in addition to its own weight.

When required — Specified in requirements supplemental to this Specification.

Work — The furnishing and performance of equipment, services, labor, and materials required by the Contract Documents for the construction of masonry for the project or part of project under consideration.

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

TMS 602 SPECIFICATION

1.3 — Reference standards

Standards referred to in this Specification are listed below with their serial designations, including year of adoption or revision, and are declared to be part of this Specification as if fully set forth in this document except as modified here.

American Concrete Institute

ACI 117-10 ~~Standard~~ Specifications for Tolerances for Concrete Construction and Materials (117-10) and Commentary-Reapproved 2015

ACI 318-19 Building Code Requirements for Structural Concrete

American Iron and Steel Institute

AISI S240-15 North American Standard for Cold-Formed Steel Structural Framing

American National Standards Institute

ANSI A118.4-19 American National Standard Specifications for Modified Dry-Set Cement Mortar

ANSI A118.15-19 American National Standard Specifications for Improved Modified Dry-Set Cement Mortar

ANSI A137.1-19 American National Standard Specifications for Ceramic Tile

American Wood Council

AWC NDS-18 National Design Specification (NDS) for Wood Construction – with 2018 NDS ~~Design~~ Supplement

ASTM International

ASTM A36/A36M-19 Standard Specification for Carbon Structural Steel

ASTM A123/A123M-17 Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products

ASTM A153/A153M-16a Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware

ASTM A240/A240M-20a Standard Specification for Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications

ASTM A307-~~14e1~~21 Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60000 PSI Tensile Strength

ASTM A416/A416M-18 Standard Specification for Low-Relaxation, Seven-Wire Steel Strand for Prestressed Concrete

ASTM A421/A421M-~~15~~21 Standard Specification for Stress-Relieved Steel Wire for Prestressed Concrete

COMMENTARY

1.3 — Reference standards

This list of standards includes material specifications, sampling, test methods, detailing requirements, design procedures, and classifications. Standards produced by ASTM International (ASTM) are referenced whenever possible. Material manufacturers and testing laboratories are familiar with ASTM standards that are the result of a consensus process. In the few cases where standards do not exist for materials or methods, the Committee developed requirements. Specific dates are given because changes to the standards alter this Specification. Many of these standards require compliance with additional standards.

Contact information for these organizations is given below:

American Concrete Institute (ACI)
38800 Country Club Drive
Farmington Hills, MI 48331
www.aci-int.org

American Iron and Steel Institute (AISI)
25 Massachusetts Avenue NW, Suite 800
Washington, DC 20001
www.steel.org

American National Standards Institute (ANSI)
25 West 43rd Street, 4th Floor
New York, NY 10036
www.ansi.org

American Wood Council (AWC)
222 Catoctin Circle SE, Suite 201
Leesburg, VA 20175
www.awc.org

ASTM International (ASTM)
100 Barr Harbor Drive
West Conshohocken, PA 19428-2959
www.astm.org

American Welding Society (AWS)
8669 NW 36th Street, Suite 130
Miami, Florida 33166-6672
www.aws.org

Federal Test Method Standard (FTMS) from:
U.S. Army General Material and Parts Center
Petroleum Field Office (East)
New Cumberland Army Depot
New Cumberland, PA 17070

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TMS 602 SPECIFICATION

COMMENTARY

1.3 — Reference standards (Continued)

ASTM A480/A480M-20a Standard Specification for General Requirements for Flat-Rolled Stainless and Heat-Resisting Steel Plate, Sheet, and Strip

ASTM A510/A510M~~18~~-20 Standard Specification for General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel, and Alloy Steel

ASTM A580/A580M-18 Standard Specification for Stainless Steel Wire

ASTM A615/A615M-20 Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement

ASTM A641/A641M-19 Standard Specification for Zinc-Coated (Galvanized) Carbon Steel Wire

ASTM A653/A653M-20 Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

ASTM A666-15 Standard Specification for Annealed or Cold-Worked Austenitic Stainless Steel Sheet, Strip, Plate, and Flat Bar

ASTM A706/A706M-16 Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement

ASTM A722/A722M-18 Standard Specification for High-Strength Steel Bars for Prestressed Concrete

ASTM A767/A767M-19 Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement

ASTM A775/A775M-19 Standard Specification for Epoxy-Coated Steel Reinforcing Bars

ASTM A884/A884M-19e1 Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement

ASTM A899-91(~~2014~~2021) Standard Specification for Steel Wire, Epoxy-Coated

ASTM A951/A951M-16e1 Standard Specification for Steel Wire for Masonry Joint Reinforcement

ASTM A996/A996M-16 Standard Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete Reinforcement

ASTM A1008/A1008M-~~20~~21a Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Required Hardness, Solution Hardened, and Bake Hardenable

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TMS 602 SPECIFICATION

1.3 — Reference standards (Continued)

ASTM A1022 / A1022M-16b Standard Specification for Deformed and Plain Stainless Steel Wire and Welded Wire for Concrete Reinforcement

ASTM A1064/A1064M-18a Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete

ASTM B117-19 Standard Practice for Operating Salt Spray (Fog) Apparatus

ASTM C34-17 Standard Specification for Structural Clay Loadbearing Wall Tile

ASTM C55-17 Standard Specification for Concrete Building Brick

ASTM C56-13 (2017) Standard Specification for Structural Clay Nonloadbearing Tile

ASTM C62-17 Standard Specification for Building Brick (Solid Masonry Units Made from Clay or Shale)

ASTM C67-~~20~~21 Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile

ASTM C73-17 Standard Specification for Calcium Silicate Brick (Sand-Lime Brick)

ASTM C90-~~16a~~21 Standard Specification for Loadbearing Concrete Masonry Units

ASTM C109/C109M-~~20b~~21 Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens)

ASTM C126-19 Standard Specification for Ceramic Glazed Structural Clay Facing Tile, Facing Brick, and Solid Masonry Units

ASTM C129-17 Standard Specification for Nonloadbearing Concrete Masonry Units

ASTM C143/C143M-20 Standard Test Method for Slump of Hydraulic-Cement Concrete

ASTM C144-18 Standard Specification for Aggregate for Masonry Mortar

ASTM C150/C150M-~~20~~21 Standard Specification for Portland Cement

ASTM C212-~~20~~21 Standard Specification for Structural Clay Facing Tile

ASTM C216-~~19~~21 Standard Specification for Facing Brick (Solid Masonry Units Made from Clay or Shale)

ASTM C270-19ae1 Standard Specification for Mortar for Unit Masonry

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TMS 602 SPECIFICATION

COMMENTARY

1.3 — Reference standards (Continued)

ASTM C476-20 Standard Specification for Grout for Masonry

ASTM C482-20 Standard Test Method for Bond Strength of Ceramic Tile to Portland Cement Paste

ASTM C503/C503M-15 Standard Specification for Marble Dimension Stone

ASTM C568/C568M-15 Standard Specification for Limestone Dimension Stone

ASTM C615/C615M-18e1 Standard Specification for Granite Dimension Stone

ASTM C616/C616M-15 Standard Specification for Quartz-Based Dimension Stone

ASTM C629/C629M-15 Standard Specification for Slate Dimension Stone

ASTM C652-~~19b~~21 Standard Specification for Hollow Brick (Hollow Masonry Units Made from Clay or Shale)

ASTM C744-~~16~~ 21 Standard Specification for Prefaced Concrete and Calcium Silicate Masonry Units

ASTM C847-18 Standard Specification for Metal Lath

ASTM C901-18 Standard Specification for Prefabricated Masonry Panels

ASTM C920-18 Standard Specification for Elastomeric Joint Sealants

ASTM C926-~~20b~~ 21 Standard Specification for Application of Portland Cement-Based Plaster

ASTM C933-18 Standard Specification for Welded Wire Lath

ASTM C1019-~~19~~ 20 Standard Test Method for Sampling and Testing Grout for Masonry

ASTM C1032-18 Standard Specification for Woven Wire Plaster Base

ASTM C1063-~~20~~ 21 Standard Specification for Installation of Lathing and Furring to Receive Interior and Exterior Portland Cement-Based Plaster

ASTM C1072-19 Standard Test Method for Measurement of Masonry Flexural Bond Strength

ASTM C1088-20 Standard Specification for Thin Veneer Brick Units Made from Clay or Shale

ASTM C1314-~~18~~ 21 Standard Test Method for Compressive Strength of Masonry Prisms

ASTM C1325-~~19~~ 21 Standard Specification for Fiber-Mat Reinforced Cementitious Backer Units

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TMS 602 SPECIFICATION

COMMENTARY

1.3 — Reference standards (Continued)

ASTM C1364-19 Standard Specification for Architectural Cast Stone

ASTM C1405-~~20a~~ 21 Standard Specification for Glazed Brick (Single Fired, Brick Units)

ASTM C1532/C1532M-~~20~~ 21 Standard Practice for Selection, Removal and Shipment of Manufactured Masonry Units and Masonry Specimens from Existing Construction

ASTM C1611/C1611M-~~18~~21 Standard Test Method for Slump Flow of Self-Consolidating Concrete

ASTM C1634-~~17~~ 20 Standard Specification for Concrete Facing Brick and Other Concrete Masonry Facing Units

ASTM C1660-10 (2018) Standard Specification for Thin-bed Mortar for Autoclaved Aerated Concrete (AAC) Masonry

ASTM C1670/C1670M-~~20a~~21a Standard Specification for Adhered Manufactured Stone Masonry Veneer Units

ASTM C1691-~~11~~ 21 ~~(2017)~~ Standard Specification for Unreinforced Autoclaved Aerated Concrete (AAC) Masonry Units

ASTM C1692-18 Standard Practice for Construction and Testing of Autoclaved Aerated Concrete (AAC) Masonry

ASTM C1693-11 (2017) Standard Specification for Autoclaved Aerated Concrete (AAC).

ASTM C1714/C1714M – 19a Standard Specification for Preblended Dry Mortar Mix for Unit Masonry

ASTM C1788-~~14 (2019)~~20 Standard Specification for Non-Metallic Plaster Bases (Lath) Used with Portland Cement Based Plaster in Vertical Wall Applications

ASTM C1877-19 Standard Specification for Adhered Concrete Masonry Units

ASTM D92-18 Standard Test Method for Flash and Fire Points by Cleveland Open Cup Tester

ASTM D95-13 (2018) Standard Test Method for Water in Petroleum Products and Bituminous Materials by Distillation

ASTM D512-12 Standard Test Methods for Chloride Ion in Water

ASTM D566-20 Standard Test Method for Dropping Point of Lubricating Grease

ASTM D610-08 (2019) Standard Practice for Evaluating Degree of Rusting on Painted Steel Surfaces

ASTM D638-14 Standard Test Method for Tensile Properties of Plastics

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TMS 602 SPECIFICATION

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1.3 — Reference standards (Continued)

ASTM D994/D994M-11 (2016) Standard Specification for Preformed Expansion Joint Filler for Concrete (Bituminous Type)

ASTM D1056-~~14~~ 20 Standard Specification for Flexible Cellular Materials — Sponge or Expanded Rubber

ASTM D1187/D1187M-97 (2018) Standard Specification for Asphalt-Base Emulsions for Use as Protective Coatings for Metal

ASTM D1227/D1227M-13 (2019)e1 Standard Specification for Emulsified Asphalt Used as a Protective Coating for Roofing

ASTM D2000-18 Standard Classification System for Rubber Products in Automotive Applications

ASTM D2265-20 Standard Test Method for Dropping Point of Lubricating Grease Over Wide Temperature Range

ASTM D2287-19 Standard Classification System and Basis for Specification for Nonrigid Vinyl Chloride Polymer and Copolymer Molding and Extrusion Compounds

ASTM D4289-19 Standard Test Method for Elastomer Compatibility of Lubricating Greases and Fluids

ASTM D7957/D7957M – 17 Standard Specification for Solid Round Glass Fiber Reinforced Polymer Bars for Concrete Reinforcement

ASTM E328-~~13~~21 Standard Test Methods for Stress Relaxation Tests for Materials and Structures

ASTM E518/E518M-~~15~~21 Standard Test Methods for Flexural Bond Strength of Masonry

ASTM F959/F959M-17a Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners, Inch and Metric Series

ASTM F1554-~~18~~20 Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength

American Welding Society

AWS D 1.4/D1.4M:2018 Structural Welding Code – Steel Reinforcing ~~Steel~~Bars

Federal Test Method Standard

FED-STD-791D (2007): Testing Method of Lubricants, Liquid Fuels, and Related Products. Federal Test Method Standard from the U.S. Army General Material and Parts Center, Petroleum Field Office (East), New Cumberland Army Depot, New Cumberland, PA 17070

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TMS 602 SPECIFICATION

1.4 — System description

1.4 A. *Compressive strength requirements* — Compressive strength of masonry in each masonry wythe and grouted collar joint shall equal or exceed the applicable f'_m or f'_{AAC}. For partially grouted masonry, the compressive strength of both the grouted and ungrouted masonry shall equal or exceed the applicable f'_m. At the transfer of prestress, the compressive strength of the masonry shall equal or exceed f'_{mi}.

1.4 B. *Compressive strength determination*

1. *Methods for determination of compressive strength* — Determine the compressive strength for each wythe by the unit strength method or by the prism test method as specified here.

2. *Unit strength method*

COMMENTARY

1.4 — System description

1.4 A. *Compressive strength requirements* — Design is based on a certain f'_m or f'_{AAC} and this compressive strength value must be achieved or exceeded. In a multiwythe wall designed as a composite wall, the compressive strength of masonry for each wythe or grouted collar joint must equal or exceed f'_m or f'_{AAC}.

1.4 B. *Compressive strength determination*

1. *Methods for determination of compressive strength* — Two methods are permitted to verify compliance with the specified compressive strength of masonry during construction: the unit strength method and the prism test method. The unit strength method has several advantages. It is less expensive than the prism test method, and it eliminates the possibility of unrepresentative low values due to errors in the construction, transport and testing of prisms. The prism test method also has advantages. Although it often requires specialized testing equipment that may not be readily available in all areas, it generally provides higher values than the unit strength method, when properly executed. Local practices and jobsite conditions may favor one method over the other.

   This Specification permits the contractor to select the method of verifying compliance with the specified compressive strength of masonry, unless a method is stipulated in the Project Specifications or Project Drawings.

2. *Unit strength method* — Compliance with the requirement for f'_m, based on the compressive strength of masonry units, grout, and mortar type, is permitted instead of prism testing.

   The influence of mortar joint thickness is noted by the maximum joint thickness. Grout strength greater than or equal to f'_m fulfills the requirements of Article 1.4 A and TMS 402 Section 3.1.3.1.

   Tables 1 and 2 can be used to establish compliance with the specified compressive strength of masonry by selecting the mortar type that will be used (column 2 or column 3) and then selecting the net area compressive strength of units that will be used within that column. If the net area compressive strength of masonry (column 1) in the corresponding row is at least as strong as the specified compressive strength of masonry, then compliance is confirmed. These tables can also be used to identify the material strengths that are required to achieve the specified compressive strength of masonry by first selecting the net area compressive strength of masonry (column 1) that

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a. *Clay masonry* — Use Table 1 to determine the compressive strength of clay masonry based on the strength of the units and the type of mortar specified, when masonry complies with the following requirements:

1) Units are sampled and tested to verify conformance with ASTM C62, ASTM C216, or ASTM C652.

2) Thickness of bed joints does not exceed $^5/_8$ in. (15.9 mm).

3) For grouted masonry, the grout conforms to Article 2.2.

COMMENTARY

is required, and then reading across the row to the combination of mortar type and unit strength (in columns 2 and 3) that will be required to achieve that net area compressive strength. In these tables, the net area compressive strengths of masonry values are conservative estimates of the expected assembly compressive strength.

The minimum compressive strength of grout is required to meet the value in the first column, or 2,000 psi, whichever is greater as required by Article 2.2 B.

a. *Clay masonry* — The original values of net area compressive strength of clay masonry in Table 1 were derived from research conducted by the Structural Clay Products Institute (SCPI (1969)).

The original values were based on testing of solid clay masonry units (SCPI (1969)) and portland cement-lime mortar. Further testing (Brown and Borchelt (1990)) has shown that the values are applicable for hollow and solid clay masonry units with all mortar types. A plot of the data is shown in Figure SC-2.

SCPI (1969) uses a height-to-thickness ratio of five as a basis to establish prism compressive strength. TMS 402 uses a different method to design for axial stress so it was necessary to change the basic prism *h/t* ratio to two. This corresponds to the *h/t* ratio used for concrete masonry in TMS 402 and for all masonry in other codes. The net effect is to increase the net area compressive strength of brick masonry as shown in Table 1 by 22 percent over that in Figure SC-2.

Table 1: Compressive strength of masonry based on the compressive strength of clay masonry units and type of mortar used in construction

Net area compressive strength of clay masonry, psi (MPa)	Net area compressive strength of clay masonry units, psi (MPa)	
	Type M or S mortar	Type N mortar
1,000 (6.90)	1,700 (11.72)	2,100 (14.48)
1,500 (10.34)	3,350 (23.10)	4,150 (28.61)
2,000 (13.79)	4,950 (34.13)	6,200 (42.75)
2,500 (17.24)	6,600 (45.51)	8,250 (56.88)
3,000 (20.69)	8,250 (56.88)	10,300 (71.02)
3,500 (24.13)	9,900 (68.26)	—
4,000 (27.58)	11,500 (79.29)	—

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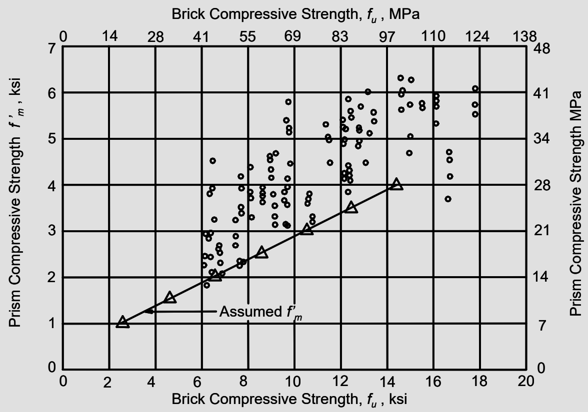
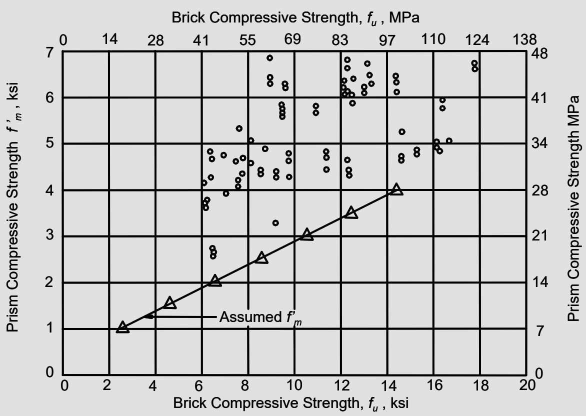


(a) Prism Strength vs. Brick Strength
(Type S Mortar, Commercial Laboratories)



(b) Prism Strength vs. Brick Strength
(Type S Mortar, SCPI Laboratory)

Figure SC-2 — Compressive strength of masonry versus clay masonry unit strength (See Commentary Article 1.4 B.2.a)

TMS 602 SPECIFICATION

1.4 B.2. *Unit strength method* (Continued)

b. *Concrete masonry* — Use Table 2 to determine the compressive strength of concrete masonry based on the strength of the unit and type of mortar specified, when masonry complies with the following requirements:

1) Units are sampled and tested to verify conformance with, ASTM C90.

2) Thickness of bed joints does not exceed $^5/_8$ in. (15.9 mm).

3) For grouted masonry, the grout conforms to Article 2.2.

COMMENTARY

b. *Concrete masonry* — Prior to the 2013 edition of this Specification, the standardized correlations between unit compressive strength, mortar type, and resulting assembly compressive strength of concrete masonry were established using prism test results collected from the 1950s through the 1980s. The result was a database of prism compressive strengths with statistically high variability, which when introduced into the Specification, drove the lower bound design values between unit, mortar, and prism to very conservative values. The reasons for the inherent historical conservatism in the unit strength table are twofold: 1) When originally introduced, the testing procedures and equipment used to develop the prism test data were considerably less refined than they are today. Changes introduced into ASTM C1314, particularly requirements for stiffer/thicker bearing platens on testing equipment, produce more consistent, repeatable compressive strength results. 2) Previous testing procedures either did not control the construction, curing, and testing of masonry prisms, or permitted many procedures for doing so. As a result, a single set of materials could produce prism test results that varied significantly depending upon how the prisms were constructed, cured, and tested. Often, a field-constructed and field-cured prism would test to a lower value than a laboratory-constructed and laboratory-cured prism. Consequently, the compressive-strength values for concrete masonry prisms used to develop historical versions of the unit strength tables are not directly comparable to the compressive-strength values that would be obtained today.

Table 2: Compressive strength of masonry based on the compressive strength of concrete masonry units and type of mortar used in construction

Net area compressive strength of concrete masonry, psi (MPa)[1]	Net area compressive strength of ASTM C90 concrete masonry units, psi (MPa)	
	Type M or S mortar	Type N mortar
1,750 (12.07)	---	2,000 (13.79)
2,000 (13.79)	2,000 (13.79)	2,650 (18.27)
2,250 (15.51)	2,600 (17.93)	3,400 (23.44)
2,500 (17.24)	3,250 (22.41)	4,350 (28.96)
2,750 (18.96)	3,900 (26.89)	-----
3,000 (20.69)	4,500 (31.03)	-----

[1] For units of less than 4 in. (102 mm) nominal height, use 85 percent of the values listed.

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1.4 B.2.b *Unit strength method* (Continued)

COMMENTARY

In 2010, the National Concrete Masonry Association (NCMA (2012)) began compiling prism test data to create a new database that would permit the development of a new unit strength table for concrete masonry that would better represent results from current prism tests. Concrete brick (ASTM C55 and ASTM C1634) are not included in Table 2 because the NCMA research program did not include these units. Most concrete brick are used in applications not requiring that f'_m be specified (such as veneer). Where f'_m is required for concrete brick applications, prism testing is required to verify the compressive strength.

The unit strength method was generated using prism test data as shown in Figures SC-3 and SC-4. The values in Table 2 are based on a consistent statistical criterion, with slight modifications based on engineering judgment.

For each specified unit strength and mortar type, the resulting masonry assembly compressive strengths were assumed to be normally distributed. Using the NCMA data for each specified unit strength and mortar type, and including the effects of sample size, the 75th percent confidence level on the 10-percentile value was calculated. That is, the value that would be expected to exceed the lower 10% fractile of the entire population 75% of the time. The criterion gives results that are reasonably consistent with other codes and standards (Bennett (2010)). Choosing the 10-percentile value results in an approximately 1% probability that the average of three prism test specimens will be less than the tabulated value.

For a given unit strength and mortar type, the resulting masonry assembly compressive strength also depends on the height of the units. The lateral expansion of the unit due to unit and mortar incompatibility increases with reduced unit height (Drysdale et al (1999)). A reduction factor in the compressive strength of masonry is required for masonry constructed of units less than 4 in. (102 mm) in nominal height, but need not be applied to masonry in which occasional units are cut to fit.

COMMENTARY

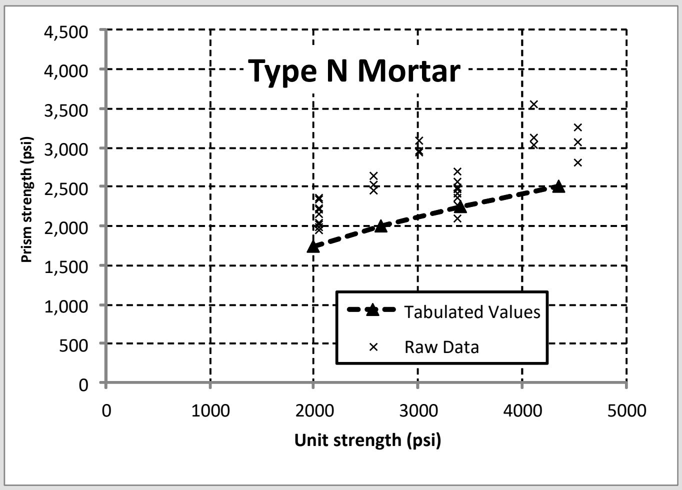

Figure SC-3 — Compressive strength of concrete masonry versus compressive strength of concrete masonry units – Type N Mortar

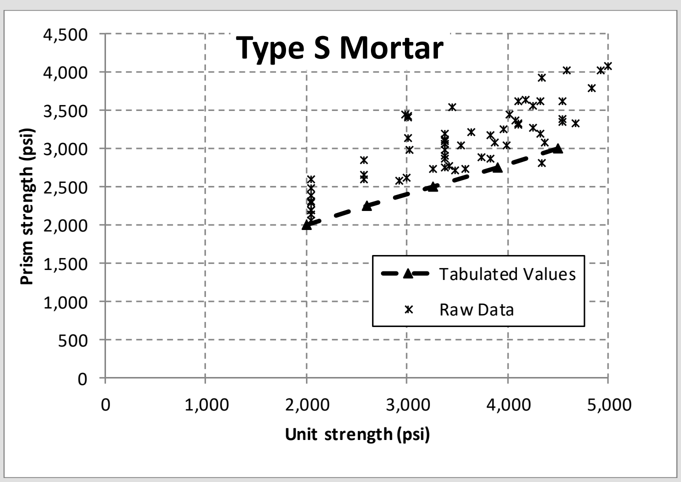

Figure SC-4 — Compressive strength of concrete masonry versus compressive strength of concrete masonry units – Type S Mortar

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1.4 B.2. *Unit strength method* (Continued)

c. *AAC masonry* — Determine the compressive strength of masonry based on the strength of the AAC masonry unit only, when masonry complies with the following requirements:

1) Units conform to Article 2.3 E.

2) Thickness of bed joints does not exceed $^1/_8$ in. (3.2 mm).

3) For grouted masonry, the grout conforms to Article 2.2.

d. *Cast stone masonry* — Determine the compressive strength of cast stone masonry using the prism test method in accordance with ASTM C1314.

3. *Prism test method* — Determine the compressive strength of clay masonry and concrete masonry by the prism test method in accordance with ASTM C1314.

COMMENTARY

c. *AAC masonry* — The strength of AAC masonry, f'_{AAC}, is controlled by the strength class of the AAC unit as defined by ASTM C1693. The strength of the thin-bed mortar and its bond in compression and shear will exceed the strength of the unit.

3. *Prism test method* — The prism test method described in ASTM C1314 was selected as a uniform method of testing clay masonry and concrete masonry to determine their compressive strengths. Masonry design is based on the compressive strength established at 28 days. The prism test method is used as an alternative to the unit strength method.

ASTM C1314 provides for testing masonry prisms at 28 days or at any designated test age. Therefore, a shorter time period, such as a 7-day test, could be used to estimate the 28-day strength based on a previously established relationship between the results of tests conducted at the shorter time period and results of the 28-day tests. Materials and workmanship of the previously established relationship must be representative of the prisms being tested.

Compliance with the specified compressive strength of masonry can be determined by the prism method instead of the unit strength method. ASTM C1314 uses materials and workmanship to construct prisms representative of those to be used in the structure. For example, strengths of different size units having the same properties, mix design, manufacturing process and curing method can be verified by testing only one of the unit sizes. Atkinson and Kingsley (1985), Priestley and Elder (1983), Miller et al (1979), Noland (1982) and Hegemier et al (1978) discuss prism testing. Many more references on the prism test method parameters and results could be added. The adoption of ASTM C1314 alleviates most of the concerns stated in the above references.

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1.4 B. *Compressive strength determination* (Continued)

4. *Testing prisms from constructed masonry* — When approved by the building official, acceptance of masonry that does not meet the requirements of Article 1.4 B.2 or 1.4 B.3 is permitted to be based on tests of prisms cut from the masonry construction.

    a. *Prism sampling and removal* — For each 5,000 square feet (465 m^2) of wall area in question, saw-cut a minimum of three prisms from completed masonry. Select, remove and transport prisms in accordance with ASTM C1532/C1532M. Determine the length, width and height dimensions of the prism and test prisms when at least 28 days old in accordance with ASTM C1314.

    b. *Compressive strength calculations* — Calculate the compressive strength of prisms in accordance with ASTM C1314.

    c. *Compliance* — Strengths determined from saw-cut prisms shall equal or exceed the specified compressive strength of masonry. Additional testing of specimens cut from construction in question is permitted.

COMMENTARY

4. *Testing prisms from constructed masonry* — While uncommon, there are times when the compressive strength of masonry determined by the unit strength method or prism test method may be questioned or may be lower than the specified strength. Because low strengths could be a result of inappropriate testing procedures or unintentional damage to the test specimens, prisms may be saw-cut from the completed masonry wall and tested. This section prescribes procedures for such tests.

    Such testing is difficult, is performed on masonry walls constructed at least 28 days before the test, and requires replacement of the sampled wall area. Therefore, concerted efforts should be taken so that strengths determined by the unit strength method or prism test method are adequate.

    a. *Prism sampling and removal* — Removal of prisms from a constructed wall requires care so that the prism is not damaged and that damage to the wall is minimal. Prisms must be representative of the wall, yet not contain any reinforcing steel, which would bias the results. As with a prism test taken during construction, a prism test from existing masonry requires three prism specimens.

    b. *Compressive strength calculations* — Compressive strength calculations from saw-cut specimens must be based on the net mortar bedded area, or the net mortar bedded area plus the grouted area for grouted prisms. The net area must be determined by the testing agency before the prism is tested.

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1.5 — Submittals

1.5 A. Obtain written acceptance of submittals prior to the use of the materials or methods requiring acceptance.

1.5 B. Submit the following:

1. Mix designs and test results
   a. One of the following for each mortar mix, excluding thin-bed mortar for AAC:
      1) Mix designs indicating type and proportions of ingredients in compliance with the proportion specification of ASTM C270, or
      2) Mix designs and mortar tests performed in accordance with the property specification of ASTM C270.
   b. One of the following for each grout mix:
      1) Mix designs indicating type and proportions of the ingredients according to the proportion requirements of ASTM C476, or
      2) Mix designs and grout strength test performed in accordance with ASTM C476, or
      3) Compressive strength tests performed in accordance with ASTM C1019, and slump flow and Visual Stability Index (VSI) as determined by ASTM C1611/C1611M.
2. Material certificates — Material certificates for the following, certifying that each material is in compliance.
   a. Reinforcement
   b. Mechanical splices
   ~~b~~c. Anchors, ties, fasteners, and metal accessories
   ~~c~~d. Masonry units
   ~~d~~e. Mortar, thin-bed mortar for AAC, and grout materials
   ~~e~~f. Self-consolidating grout
   ~~f~~g. Lath, scratch coat and setting bed mortar
3. Construction procedures
   a. Cold weather construction procedures
   b. Hot weather construction procedures
4. Manufacturer's instructions, including placement tolerances
   a. Adjustable wall ties
   b. Adjustable veneer anchors
5. Qualifications
   a. Field testing personnel

COMMENTARY

1.5 — Submittals

Submittals and their subsequent acceptance or rejection on a timely basis will keep the project moving smoothly. If the specifier wishes to require a higher level of quality assurance than the minimum required by this Specification, submittals may be required for one or more of the following: shop drawings for reinforced masonry and lintels; sample specimens of masonry units, colored mortar, each type of movement joint accessory, anchor, tie, fastener, and metal accessory; and test results for masonry units, mortar, reinforcement, and grout. Shop drawings illustrating masonry unit shape and cross-sectional dimensions may be desirable if the designer needs to verify compliance with the maximum reinforcement requirements of TMS 402 Section 6.1.3.2.4 instead of using the alternative maximum reinforcement requirements of TMS 402 Section 6.1.3.2.5.1 or Section 6.1.3.2.5.2.

5. Qualifications — Use of qualified testing and inspection personnel helps minimize problems with test results that can delay projects or add

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b. Lab testing personnel

c. Special Inspector

6. When more than one reinforcing bar will be placed in a closed-bottom masonry unit, provide a drawing showing the dimensions of the vertical cross-section of the closed-bottom unit proposed to be used for the project.

1.6 — Quality assurance

1.6 A. *Testing Agency's services and duties*

1. Utilize qualified field testing technicians to observe or perform the preparation and handling of grout

COMMENTARY

unnecessary costs. Testing technicians and Special Inspectors can become qualified by various national certification programs.

6. Some applications of closed-bottom masonry units are for bond beams and lintels. Closed-bottom unit drawings will provide the information needed by the Architect/Engineer to verify that the as-designed reinforcement quantity and location can fit and will meet the design limitation on reinforcement percentage in TMS 402 Section 6.1.3.2.4.

1.6 — Quality assurance

Quality assurance consists of the actions taken by an owner or owner's representative, including establishing the quality assurance requirements, to provide assurance that materials and workmanship are in accordance with the contract documents. Quality assurance includes quality control measures as well as testing and inspection to verify compliance. The term quality control was not used in this Specification because its meaning varies with the perspective of the parties involved in the project.

The owner and Architect/Engineer may require a testing laboratory to provide some or all of the verification requirements mentioned in Table 3. Item 4 of Table 4 provides criteria for preparation of test specimens.

The quality objectives are met when the building is properly designed, completed using materials complying with product specifications using adequate construction practices, and is adequately maintained. Special Inspection and testing are important components of the quality assurance program, which is used to meet the objective of quality in construction.

1.6 A. *Testing Agency's services and duties* — Implementation of testing and inspection requirements contained in the Quality Assurance Tables requires detailed knowledge of the appropriate procedures. Comprehensive (Chrysler (~~2010~~2017); NCMA (2008); BIA TN 39 (2001); BIA TN 39B (1988)) and summary (SCI and MIA (2006)) testing and inspection procedures are available from recognized industry sources which may be referenced for assistance in complying with the specified Quality Assurance program.

ASTM C1093 defines the duties and responsibilities of testing agency personnel and defines the technical requirements for equipment used in testing masonry materials. Testing agencies who are accredited or inspected for conformance to the requirements of ASTM C1093 by a recognized evaluation authority are qualified to test masonry.

1. Field technicians who are certified in accordance with the requirements of ACI Masonry Field Testing Technician Certification Program, or an

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specimens, mortar specimens and / or masonry prisms.

2. Utilize qualified laboratory technicians to perform required laboratory tests.

1.6 A. *Testing Agency's services and duties* (Continued)

2. Sample and test in accordance with Tables 3 and 4 as specified for the project.
3. Unless otherwise required, report test results to the Architect/Engineer, Inspection Agency, and Contractor promptly after they are performed. Include in test reports a summary of conditions under which test specimens were stored prior to testing and state what portion of the construction is represented by each test.
4. When there is reason to believe that any material furnished or work performed by the Contractor fails to fulfill the requirements of the Contract Documents, report such discrepancy to the Architect/Engineer, Inspection Agency, and Contractor.
5. Unless otherwise required, the Owner will retain the Testing Agency.

COMMENTARY

equivalent program, are qualified to observe and / or prepare masonry specimens.

2. Masonry testing laboratory personnel who are certified in accordance with ACI Masonry Laboratory Testing Technician Certification Program, or equivalent program, are qualified.

Table 3: Minimum Verification Requirements

Minimum Verification	Required for Quality Assurance[a] Level 1	Level 2	Level 3	Reference for Criteria TMS 602
Prior to construction, verification of compliance of submittals.	R	R	R	Art. 1.5
Prior to construction, verification of f'_m and f'_{AAC}, except where specifically exempted by the Code.	NR	R	R	Art. 1.4 B
During construction, verification of Slump flow and Visual Stability Index (VSI) when self-consolidating grout is delivered to the project site.	NR	R	R	Art. 1.5 & 1.6.3
During construction, verification of f'_m, f'_{mi}, and f'_{AAC} for every 5,000 sq. ft. (465 sq. m).	NR	NR	R	Art. 1.4 B
During construction, verification of proportions of materials as delivered to the project site for premixed or preblended mortar, prestressing grout, and grout other than self-consolidating grout.	NR	NR	R	Art. 1.4 B

(a) R=Required, NR=Not Required

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Table 4: Minimum Special Inspection Requirements

MINIMUM SPECIAL INSPECTION					
Inspection Task	**Frequency [a]**			**Reference for Criteria**	
	Level 1	**Level 2**	**Level 3**	**TMS 402**	**TMS 602**
1. As masonry construction begins, verify that the following are in compliance:					
a. Proportions of site-prepared mortar	NR	P	P		Art. 2.1 & 2.6 A & C
b. Grade and size of prestressing tendons and anchorages	NR	P	P		Art. 2.4 B & 2.4 ~~K~~ M & ~~L~~N
c. Grade, type and size of reinforcement, connectors, and anchor bolts	NR	P	P		Art. 2.4 A, D, E, F, G, H, I, ~~I~~J, K, &
d. Prestressing technique	NR	P	P		Art. 3.6 B
e. Properties of thin-bed mortar for AAC masonry	NR	C[b]/P[c]	C		Art. 2.1 DC.1
f. Sample panel construction	NR	P	C		Art. 1.6 D
2. Prior to grouting, verify that the following are in compliance:					
a. Grout space	NR	P	C		Art. 3.2 D & 3.2 F
b. Placement of prestressing tendons and anchorages	NR	P	P	Sec. 10.8 & 10.9	Art. 3.6
c. Placement of reinforcement, connectors, and anchor bolts	NR	P	C	Sec. 6.1, 6.3.1, 6.3.6, & 6.3.7	Art. 3.2 E & 3.4
d. Proportions of site-prepared grout and prestressing grout for bonded tendons	NR	P	P		Art. 2.6 B & 2.4 ~~K~~M.1.b
3. Verify compliance of the following during construction:					
a. Materials and procedures with the approved submittals	NR	P	P		Art. 1.5
b. Placement of masonry units and mortar joint construction	NR	P	P		Art. 3.3 B
c. Size and location of structural members	NR	P	P		Art. 3.3 G
d. Type, size, and location of anchors, including other details of anchorage of masonry to structural members, frames, or other construction	NR	P	C	Sec. 1.2.1(e), 6.2.1, & 6.3.1	
e. Type, size, and location of veneer ties & movement joints	P[d]	P[d]	--	Sec. 13.2	Art. 3.4 ~~E~~D
f. Installation of adhered veneer	P[d]	P[d]	--	Sec. 13.3	Art. 3.3 D
g. Welding of reinforcement	NR	C	C	Sec.6.1.7.3	
h. Preparation, construction, and protection of masonry during cold weather (temperature below 40°F (4.4°C)) or hot weather (temperature above 90°F (32.2°C))	NR	P	P		Art. 1.8 C & D
i. Application and measurement of prestressing force	NR	C	C		Art. 3.6 B
h. Placement of grout	NR	C	C		Art. 3.5
j. Placement of prestressing grout for bonded tendons	NR	C	C		3.6 C
k Placement of AAC masonry units and construction of thin-bed mortar joints	NR	C[b]/P[c]	C		Art. 3.3 B.8 & 3.3 G.1.b
4. Observe preparation of grout specimens, mortar specimens, and/or prisms	NR	P	C		Art. 1.4 B.2.a.3, B.2.b.3, B.2.c.3, B.3, & B.4

(a) Frequency refers to the frequency of inspection, which may be continuous during the listed task or periodically during the listed task, as defined in the table. NR=Not Required, P=Periodic, C=Continuous

(b) Required for the first 5000 square feet (465 square meters) of AAC masonry.

(c) Required after the first 5000 square feet (465 square meters) of AAC masonry.

(d) Periodic inspection of veneers is required when the height of the veneer exceeds 60 ft (18.3 m) above grade plane.

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1.6 B. *Inspection Agency's services and duties*

1. Utilize qualified Special Inspectors to inspect and evaluate construction. Inspect and evaluate in accordance with Tables 3 and 4, as specified for the project.
2. When required, inspect and evaluate items beyond the scope of the applicable QA Table.
3. Unless otherwise required, report inspection results to the Architect/Engineer, and Contractor promptly after they are performed. Include in inspection reports a summary of conditions under which the inspections were made and state what portion of the construction is represented by each inspection.
4. Furnish inspection reports to the Architect/Engineer and Contractor.
5. When there is reason to believe that any material furnished or work performed by the Contractor fails to fulfill the requirements of the Contract Documents, report such discrepancy to the Architect/Engineer and to the Contractor and, when requested, to the Owner and the Building Official.
6. Submit a final signed report stating whether the Work requiring Special Inspection was, to the best of the Inspection Agency's knowledge, in conformance. Submit the final report to the Architect/Engineer and Contractor.
7. Unless otherwise required, the Owner will retain the Inspection Agency.

COMMENTARY

1.6 B. *Inspection Agency's services and duties* — TMS 402 and this Specification require that masonry be inspected. The design provisions used in TMS 402 are based on the premise that the work will be inspected, and that quality assurance measures will be implemented. Minimum verification and minimum Special Inspection requirements are given in Specification Tables 3 and 4. The Architect/Engineer may increase the amount of verification and inspection required. Certain applications, such as Masonry Veneer (Chapter 13), and Masonry Partition Walls (Chapter 15), do not require compressive strength verification of masonry as indicated in Table 3. The method of payment for inspection services is usually addressed in general conditions or other contract documents and usually is not governed by this article.

1. Special inspectors who are certified for this service by International Code Council, or other acceptable agency, are qualified.
2. Certain items such as location and conformance of wall penetrations, embedded items and flashing may not be listed in the QA Tables, but may require inspection so that the Architect/Engineer is satisfied that the installation conforms to the design.
5. This Specification requires testing reports to be distributed to the parties responsible for the design, construction, and approval of the Work. Such distribution of test reports should be indicated in contracts for inspection and testing services. Prompt distribution of testing reports allows for timely identification of either compliance or the need for corrective action. A complete record of testing allows the appropriate corrective actions for future work.

TMS 602 SPECIFICATION

1.6 C. *Contractor's services and duties*

1. Permit and facilitate access to the construction sites and the performance of activities for quality assurance by the Testing and Inspection Agencies.
2. The use of testing and inspection services does not relieve the Contractor of the responsibility to furnish materials and construction in full compliance.
3. To facilitate testing and inspection, comply with the following:
   a. Furnish necessary labor to assist the designated testing agency in obtaining and handling samples at the Project.
   b. Advise the designated Testing Agency and Inspection Agency sufficiently in advance of operations to allow for completion of quality assurance measures and for the assignment of personnel.
   c. Provide masonry materials required for preconstruction and construction testing.
4. Provide and maintain adequate facilities for the sole use of the testing agency for safe storage and proper curing of test specimens on the Project Site.
5. In the submittals, include the results of testing performed to qualify the materials and to establish mix designs.

1.6 D. *Sample panels*

1. For masonry governed by Level 2 or 3 Quality Assurance (Table 4), construct sample panels of masonry walls.
   a. Use materials and procedures accepted for the Work.
   b. The minimum sample panel dimensions are 4 ft by 4 ft (1.22 m by 1.22 m).
2. The acceptable standard for the Work is established by the accepted panel.
3. Retain sample panels at the project site until Work has been accepted.

1.6 E. *Grout demonstration panel* — Prior to masonry construction, construct a grout demonstration panel if proposed grouting procedures, construction techniques, or grout space geometry do not conform to the applicable requirements of Articles 3.5 C, 3.5 D, and 3.5 E.

COMMENTARY

1.6 C. *Contractor's services and duties* — The contractor establishes mix designs, the source for supply of materials, and suggests change orders.

The listing of duties of the inspection agency, testing agency, and contractor provide for a coordination of their tasks and a means of reporting results. The contractor is bound by contract to supply and place the materials required by the contract documents. Perfection is obviously the goal, but factors of safety included in the design method recognize that some deviation from perfection will exist. Engineering judgment must be used to evaluate reported discrepancies. Tolerances listed in Article 3.3 G were established to assure structural performance and were not based on aesthetic criteria.

1.6 D. *Sample panels* — Sample panels should contain the full range of unit and mortar color. Each procedure, including cleaning and application of coatings and sealants, should be demonstrated on the sample panel. The effect of these materials and procedures on the masonry can then be determined before large areas are treated. Because it serves as a comparison of the finished work, the sample panel should be maintained until the work has been accepted. Certain components of sample panels, such as the type of mortar joint, can have structural implications with the performance of masonry. Construct sample panels within the tolerances of Article 3.3 F. The specifier has the option of permitting a segment of the masonry construction to serve as a sample panel or requiring a separate stand-alone panel.

TMS 602 SPECIFICATION

1.7 — Delivery, storage, and handling

1.7 A. Do not use damaged masonry units, damaged components of structure, or damaged packaged material.

1.7 B. Protect cementitious materials for mortar and grout from precipitation and groundwater.

1.7 C. Do not use masonry materials that are contaminated.

1.7 D. Store different aggregates separately.

1.7 E. Protect reinforcement, ties, and metal accessories from permanent distortions and store them off the ground.

1.7 F. *GFRP reinforcing bars*

1. When handling GFRP reinforcement, use equipment that avoids damaging or abrading the GFRP reinforcing bar. Do not drop or drag GFRP reinforcement.
2. Store GFRP reinforcing bars above the surface of the ground on platforms, skids, or other supports near the point of placement. If stored outdoors for more than two months, cover GFRP reinforcing bars with opaque plastic or other types of cover that protect the bars from ultra-violet rays.
3. Prevent exposure of GFRP reinforcing bars to temperature above 120°F (48.9°C) during storage.
4. Do not use GFRP reinforcing bars with visible fibers (other than at cut ends) or any cut or defect greater than 0.04 in. (1.02 mm) deep. Repair visible damage to GFRP reinforcing bars exceeding 2 percent of surface area per foot of bar.

1.8 — Project conditions

1.8 A. *Construction loads* — Do not apply construction loads that exceed the safe superimposed load capacity of the masonry and shores, if used.

1.8 B. *Masonry protection* — Cover top of unfinished masonry work to protect it from moisture intrusion.

COMMENTARY

1.7 — Delivery, storage, and handling

The performance of masonry materials can be reduced by contamination by dirt, water, and other materials during delivery or at the project site.

Reinforcement and metal accessories are less prone than masonry materials to damage from handling.

1.8 — Project conditions

1.8 B. *Masonry protection* — Many geographic areas are subject to unpredictable weather. Masonry under construction needs to be protected from detrimental moisture intrusion, particularly when there is a possibility of freezing temperatures. In areas where dry weather is consistent, covering walls to protect against moisture intrusion during the normal progress of construction may not be required.

TMS 602 SPECIFICATION

1.8 C. *Cold weather construction* — When ambient air temperature is below 40°F (4.4°C), implement cold weather procedures and comply with the following:

1. Do not lay glass unit masonry.
2. *Preparation* — Comply with the following requirements prior to conducting masonry work:
   a. Do not lay masonry units having either a temperature below 20°F (-6.7°C) or containing frozen moisture, visible ice, or snow on their surface.
   b. Remove visible ice and snow from the top surface of existing foundations and masonry to receive new construction. Heat these surfaces above freezing, using methods that do not result in damage.

COMMENTARY

1.8 C. *Cold weather construction* — The procedure described in this article represents the Committee's consensus of current good construction practice and has been framed to generally agree with the Masonry Industry Council (1999).

The provisions of Article 1.8 C are mandatory, even if the procedures submitted under Article 1.5 B.3.a are not required. The contractor has several options to achieve the results required in Article 1.8 C. The options are available because of the climatic extremes and their duration. When the air temperature at the project site or unit temperatures fall below 40°F (4.4°C), the cold weather protection plan submitted becomes mandatory. Work stoppage may be justified if a short cold spell is anticipated. Enclosures and heaters can be used as necessary.

Temperature of the masonry mortar may be measured using a metal tip immersion thermometer inserted into a sample of the mortar. The mortar sample may be mortar as contained in the mixer, in hoppers for transfer to the working face of the masonry or as available on mortar boards currently being used. The critical mortar temperatures are the temperatures at the mixer and mortar board locations. The ideal mortar temperature is 60°F to 80°F (15.6°C to 26.7°C).

Temperature of the masonry unit may be measured using a metallic surface contact thermometer. Temperature of the units may be below the ambient temperature if the requirements of Article 1.8 C.2.a are met.

The contractor may choose to enclose the entire area rather than make the sequential materials conditioning and protection modifications. Ambient temperature conditions apply while work is in progress. Minimum daily temperatures apply to the time after grouted masonry is placed. Mean daily temperatures apply to the time after ungrouted masonry is placed.

Grout made with Type III portland cement gains strength more quickly than grout mixed with Type I portland cement. This faster strength gain eliminates the need to protect masonry for the additional 24 hr period.

Construction experience, though not formally documented, suggests that AAC thin-bed mortar reaches full strength significantly faster than masonry mortar; however, it is more sensitive to cold weather applications. AAC masonry also holds heat considerably longer than concrete masonry. Cold weather requirements are therefore different for thin-bed mortar applications as compared to conventional mortar. Cold weather requirements for leveling course mortar and grout remain the same as for other masonry products.

TMS 602 SPECIFICATION

1.8 C. *Cold weather construction* (Continued)

3. *Construction* — These requirements apply to work in progress and are based on ambient air temperature. Do not heat water or aggregates used in mortar or grout above 140°F (60°C). Comply with the following requirements when the following ambient air temperatures exist:

a. 40°F to 32°F (4.4°C to 0°C):

1) Heat sand or mixing water to produce mortar temperature between 40°F (4.4°C) and 120°F (48.9°C) at the time of mixing.

2) Heat grout materials when the temperature of the materials is below 32°F (0°C).

b. Below 32°F to 25°F (0°C to -3.9°C):

1) Heat sand and mixing water to produce mortar temperature between 40°F (4.4°C) and 120°F (48.9°C) at the time of mixing. Maintain mortar temperature above freezing until used in masonry.

2) Heat grout aggregates and mixing water to produce grout temperature between 70°F (21.1°C) and 120°F (48.9°C) at the time of mixing. Maintain grout temperature above 70°F (21.1°C) at the time of grout placement.

3) Heat AAC units to a minimum temperature of 40°F (4.4°C) before installing thin-bed mortar.

c. Below 25°F to 20°F (-3.9°C to –6.7°C): Comply with Article 1.8 C.3.b and the following:

1) Heat masonry surfaces under construction to a minimum temperature of 40°F (4.4°C).

2) Use wind breaks or enclosures when the wind velocity exceeds 15 mph (24 km/hr).

3) Heat masonry to a minimum temperature of 40°F (4.4°C) prior to grouting.

d. Below 20°F (-6.7°C): Comply with Article 1.8 C.3.c and the following: Provide an enclosure and auxiliary heat to maintain air temperature above 32°F (0°C) within the enclosure.

COMMENTARY

2) Grout should be mixed to a temperature above the minimum mixing temperature to account for possible heat loss while transporting it between the mixing station and work area.

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COMMENTARY

1.8 C *Cold weather construction* (Continued)

4. *Protection* — These requirements apply after masonry is placed and are based on anticipated minimum daily temperature for grouted masonry and anticipated mean daily temperature for ungrouted masonry. Protect completed masonry in the following manner:

   a. Maintain the temperature of glass unit masonry above 40°F (4.4°C) for the first 48 hr after construction.

   b. Maintain the temperature of AAC masonry above 32°F (0°C) for the first 4 hr after thin-bed mortar application.

   c. 40°F to 25°F (4.4°C to -3.9°C): Protect newly constructed masonry by covering with a weather-resistive membrane for 24 hr after being completed.

   d. Below 25°F to 20°F (-3.9°C to -6.7°C): Cover newly constructed masonry completely with weather-resistive insulating blankets, or equal protection, for 24 hr after completion of work. Extend time period to 48 hr for grouted masonry, unless the only cement in the grout is Type III portland cement.

   e. Below 20°F (-6.7°C): Maintain newly constructed masonry temperature above 32°F (0°C) for at least 24 hr after being completed by using heated enclosures, electric heating blankets, infrared lamps, or other acceptable methods. Extend time period to 48 hr for grouted masonry, unless the only cement in the grout is Type III portland cement.

TMS 602 SPECIFICATION

1.8 D. *Hot weather construction* — Implement approved hot weather procedures and comply with the following provisions:

1. *Preparation* — Prior to conducting masonry work:

   a. When the ambient air temperature exceeds 100°F (37.8°C), or exceeds 90°F (32.2°C) with a wind velocity greater than 8 mph (12.9 km/hr):

      1) Maintain sand piles in a damp, loose condition.

      2) Provide necessary conditions and equipment to produce mortar having a temperature below 120°F (48.9°C).

      3) Fog spray or wet scratch coats until damp prior to installation of adhered veneer.

   b. When the ambient temperature exceeds 115°F (46.1°C), or exceeds 105°F (40.6°C) with a wind velocity greater than 8 mph (12.9 km/hr), implement the requirements of Article 1.8 D.1.a and shade materials and mixing equipment from direct sunlight.

2. *Construction* — While masonry work is in progress:

   a. When the ambient air temperature exceeds 100°F (37.8°C), or exceeds 90°F (32.2°C) with a wind velocity greater than 8 mph (12.9 km/hr):

      1) Maintain temperature of mortar and grout below 120°F (48.9°C).

      2) Flush mixer, mortar transport container, and mortar boards with cool water before they come into contact with mortar ingredients or mortar.

      3) Maintain mortar consistency by retempering with cool water.

      4) Use mortar within 2 hr of initial mixing.

      5) Spread thin-bed mortar no more than four feet ahead of AAC masonry units.

      6) Set AAC masonry units within one minute after spreading thin-bed mortar.

   b. When the ambient temperature exceeds 115°F (46.1°C), or exceeds 105°F (40.6°C) with a wind velocity greater than 8 mph (12.9 km/hr), implement the requirements of Article 1.8 D.2.a and use cool mixing water for mortar and grout. Ice is permitted in the mixing water prior to use. Do not permit ice in the mixing water when added to the other mortar or grout materials.

3. *Protection* — When the mean daily temperature exceeds 100°F (37.8°C) or exceeds 90°F (32.2°C) with a wind velocity greater than 8 mph (12.9 km/hr), fog spray newly constructed masonry until damp, at least three times a day until the masonry is three days old.

COMMENTARY

1.8 D. *Hot weather construction* — High temperature and low relative humidity increase the rate of moisture evaporation. These conditions can lead to “dryout” (drying of the mortar or grout before sufficient hydration has taken place) of the mortar and grout (Tomasetti (1990)). Dryout adversely affects the properties of mortar and grout because dryout signals improper curing and associated reduction of masonry strength development. The preparation, construction, and protection requirements in the Specification are minimum requirements to avoid dryout of mortar and grout and to allow for proper curing. They are based on industry practice (BIA (~~1992~~2018); Farny et al (2008), PCA (~~1993~~2006)~~; Panarese et al (1991)~~). More stringent and extensive hot weather practices may be prudent where temperatures are high, winds are strong, and humidity is low.

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During hot weather, shading masonry materials and equipment reduces mortar and grout temperatures. Scheduling construction to avoid hotter periods of the day should be considered.

See Commentary to Article 2.1 for considerations in selecting mortar materials. The most effective way of reducing mortar and grout batch temperatures is by using cool mixing water. Small batches of mortar are preferred over larger batches to minimize drying time on mortar boards. Mortar should not be used after a maximum of 2 hr after initial mixing in hot weather conditions. Use of cool water to retemper, when tempering is permitted, restores plasticity and reduces the mortar temperature (IMI (1973); BIA (~~1992~~2018); PCA (~~1993~~2006)).

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Most mason’s sand is delivered to the project in a damp, loose condition with a moisture content of about 4 to 6 percent. Sand piles should be kept cool and in a damp, loose condition by sprinkling and by covering with a plastic sheet to limit evaporation.

Research suggests that covering and moist curing of concrete masonry walls dramatically improves flexural bond strength compared to walls not covered or moist cured (NCMA (1994)).

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PART 2 — PRODUCTS

TMS 602 SPECIFICATION

2.1 — Mortar materials

2.1 A. Provide mortar of the type and color specified, and conforming with ASTM C270 or ASTM C1714.

COMMENTARY

2.1 — Mortar materials

ASTM C270 contains standards for materials used to make mortar. Thus, component material specifications need not be listed. The Architect/Engineer may wish to include only certain types of materials, or exclude others, to gain better control.

Mortars specified via ASTM C1714 / C1714M have materials and design requirements governed by ASTM C270, but are preblended in a factory instead of produced from individual raw materials delivered to the jobsite.

Certain applications may require specific mortar types such as masonry veneer, parapets or below grade masonry. ASTM C270 provides a guide for the selection of masonry mortars to assist in choosing the appropriate mortar for various applications.

There are two methods of specifying mortar under ASTM C270: proportion and property. The proportion specification directs the contractor to mix the materials in the volumetric proportions given in ASTM C270. These are repeated in Table SC-1. The property specification instructs the contractor to develop a mortar mix that will yield the specified properties under laboratory testing conditions. Table SC-2 contains the required results outlined in ASTM C270. The results are submitted to the Architect/Engineer and the mix proportions developed in the laboratory are maintained in the field. Water added in the field is determined by the mason for both methods of specifying mortar. A mortar mixed in accordance with the proportion requirements of Table SC-1 may have different physical properties than of a mortar of the same type (i.e. Type M, S, N, or O) mixed in accordance with proportions established by laboratory testing to meet the property specification requirements of Table SC-2. Higher lime content increases workability and water retentivity. ASTM C270 has an Appendix with information that can be useful in selecting mortar.

Either proportions or properties, but not both, should be specified. A good rule of thumb is to specify the weakest mortar that will perform adequately, not the strongest. Excessive amounts of pigments used to achieve mortar color may reduce both the compressive and bond strength of the masonry. Conformance to the maximum percentages indicated will limit the loss of strength to acceptable amounts. Due to the fine particle size, the water demand of the mortar increases when coloring pigments are used. Admixtures containing excessive amounts of chloride ions are detrimental to steel items placed in mortar or grout.

ASTM C270 specifies mortar testing under laboratory conditions only for acceptance of mortar mixes under the property specifications. Field sampling and testing of

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Commented [PJS60]: 20-CR-007 with editorial revisions mortar is conducted under ASTM C780 and is used to verify consistency of materials and procedures, not mortar strength. ASTM C1586 provides guidance on appropriate testing of mortar for quality assurance.

COMMENTARY

Table SC-1: ASTM C270 mortar proportion specification requirements

Mortar	Type	Proportions by volume (cementitious materials)								Aggregate ratio (measured in damp, loose conditions)
		Portland cement or blended cement	Mortar cement			Masonry cement			Hydrated lime or lime putty	
			M	S	N	M	S	N		
Cement-lime	M	1	-	-	-	-	-	-	¼	Not less than 2¼ and not more than 3 times the sum of the separate volumes of cementitious materials.
	S	1	-	-	-	-	-	-	over ¼ to ½	
	N	1	-	-	-	-	-	-	over ½ to 1¼	
	O	1	-	-	-	-	-	-	over 1¼ to 2½	
Mortar cement	M	1	-	-	1	-	-	-	-	
	M	-	1	-	-	-	-	-	-	
	S	½	-	-	1	-	-	-	-	
	S	-	-	1	-	-	-	-	-	
	N	-	-	-	1	-	-	-	-	
	O	-	-	-	1	-	-	-	-	
Masonry cement	M	1	-	-	-	-	-	1	-	
	M	-	-	-	-	1	-	-	-	
	S	½	-	-	-	-	-	1	-	
	S	-	-	-	-	-	1	-	-	
	N	-	-	-	-	-	-	1	-	
	O	-	-	-	-	-	-	1	-	

Two air entraining materials shall not be combined in mortar.

Table SC-2: ASTM C270 property specification requirements for laboratory prepared mortar

Mortar	Type	Average compressive strength at 28 days, psi (MPa)	Water retention min, percent	Air content max, percent	Aggregate ratio (measured in damp, loose conditions)
Cement-lime	M	2500 (17.2)	75	12	Not less than 2¼ and not more than 3½ times the sum of the separate volumes of cementitious materials
	S	1800 (12.4)	75	12	
	N	750 (5.2)	75	14[1]	
	O	350 (2.4)	75	14[1]	
Mortar cement	M	2500 (17.2)	75	12	
	S	1800 (12.4)	75	12	
	N	750 (5.2)	75	14[1]	
	O	350 (2.4)	75	14[1]	
Masonry cement	M	2500 (17.2)	75	18	
	S	1800 (12.4)	75	18	
	N	750 (5.2)	75	20[2]	
	O	350 (2.4)	75	20[2]	

[1] When structural reinforcement is incorporated in cement-lime or mortar cement mortar, the maximum air content shall be 12 percent.

[2] When structural reinforcement is incorporated in masonry cement mortar, the maximum air content shall be 18 percent.

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2.1 B. *Adhered veneer setting bed mortar* — Provide setting bed mortar conforming to ANSI A118.4 or A118.15.

2.1 C. *Glass unit masonry* — For exterior glass unit masonry, provide Type S mortar that conforms to Article 2.1 A. For interior glass unit masonry, provide Type S or N mortar that conforms to Article 2.1 A.

2.1 D. *AAC masonry*

1. Provide thin-bed mortar that conforms to ASTM C1660. Additional testing to verify AAC masonry assembly properties shall be conducted by the thin-bed mortar manufacturer and confirmed by an independent testing agency.
   a. Determine flexural strength in accordance with ASTM C1692. Meet the minimum flexural tensile strength values of Code Section 11.1.8.3.
   b. Determine shear strength in accordance ASTM C1692. Meet the minimum shear strength values of Code Section 11.1.8.4.
2. Mortar for leveling course shall be Type M or S. Conform to the requirements of Article 2.1 A.

2.2 — Grout materials

2.2 A. Unless otherwise required, provide grout that conforms to the requirements of ASTM C476.

2.2 B. When f'_m exceeds 2,000 psi (13.79 MPa), provide grout compressive strength that equals or exceeds f'_m. Determine compressive strength of grout in accordance with ASTM C1019.

2.2 C. Do not use admixtures unless acceptable. Field addition of admixtures is not permitted in self-consolidating grout.

COMMENTARY

2.1 C. *Glass unit masonry* — Manufacturers of glass units recommend using mortar containing a water-repellent admixture or a cement containing a water-repellent addition (Pittsburgh Corning (1992); Glashaus (1992); Beall (1989)). A workable, highly water-retentive mortar is recommended for use when conditions of high heat and low relative humidity exist during construction.

2.2 — Grout materials

ASTM C476 contains standards for materials used to make grout. Thus, component material specifications need not be listed.

Admixtures for grout include those to increase flow and to reduce shrinkage. Because self-consolidating grouts include admixtures and are delivered to the project site premixed or preblended and certified by the manufacturer, the addition of admixtures in the field is not permitted.

Self-consolidating grout meets the material requirements in ASTM C476. Because the mix is highly fluid, traditional slump cone tests for masonry grout are not applicable. The material is qualified by measuring its slump flow and determining its Visual Stability Index (VSI) using ASTM C1611/C1611M.

Because the strength of AAC units never approaches 2,000 psi (13.79 MPa), and f'_{AAC} cannot be designed above the unit material strength, AAC masonry only requires grout that meets the requirements of Article 2.2 A.

This article does not apply to prestressing grout; see Article 2.4 ~~K~~M.1.b. This article also does not apply to lightweight grout which currently lacks an ASTM standard and behavioral research.

TMS 602 SPECIFICATION

2.3 — Masonry unit materials

2.3 A. Provide concrete masonry units that conform to ASTM C55, C73, C90, C129, C744, C1634, or C1877 as specified.

COMMENTARY

2.3 — Masonry unit materials

2.3 A. Concrete masonry units are made from lightweight and normal weight aggregate, water, and cement. The units are available in a variety of shapes, sizes, colors, and strengths. Because the properties of the concrete vary with the aggregate type and mix proportions, there is a range of physical properties and weights available in concrete masonry units.

Masonry units are selected for the use and appearance desired, with minimum requirements addressed by each respective ASTM standard. When particular features are desired such as surface textures for appearance or bond, finish, color, or particular properties such as weight classification, higher compressive strength, fire resistance, thermal or acoustical performance, these features should be specified separately by the purchaser. Local suppliers should be consulted as to the availability of units having the desired features.

ASTM C73 designates sand-lime brick as either Grade SW or Grade MW. Grade SW brick are intended for use where they will be exposed to freezing temperatures in the presence of moisture. Grade MW brick are limited to applications in which they may be subjected to freezing temperature but in which they are unlikely to be saturated with water.

Table SC-3 summarizes the requirements for various concrete masonry units given in the referenced standards.

ASTM C744 addresses the properties of units with a resin facing. The units must meet the requirements of one of the other referenced standards.

Table SC-3: Concrete masonry unit requirements

ASTM Specification	Unit	Strength	Weight	Type	Grade
C55	Concrete brick	yes	yes	no	no
C73	Sand-lime brick	yes	no	no	yes
C90	Load-bearing units	yes	yes	no	no
C129	Non-load-bearing units	yes	yes	no	no
C744	Prefaced units	—	—	—	—
C1634	Concrete facing brick	yes	yes	no	no

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2.3 B. Provide clay or shale masonry units that conform to ASTM C34, C56, C62, C126, C212, C216, C652, C1088, or C1405 or to ANSI A 137.1, as specified.

COMMENTARY

2.3 B. Clay or shale masonry units are formed from those materials and referred to as brick or tile. Clay masonry units may be molded, pressed, or extruded into the desired shape. Physical properties depend upon the raw materials, the method of forming, and the firing temperature. Incipient fusion, a melting and joining of the clay particles, is necessary to develop the strength and durability of clay masonry units. A wide variety of unit shapes, sizes, colors, and strengths is available.

The intended use determines which standard specification is applicable. Generally, brick units are smaller than tile, tile is always cored, and brick may be solid or cored. Clay brick is normally exposed in use, but clay tile is usually not exposed. Grade or class is determined by exposure condition and has requirements for durability, usually given by compressive strength and absorption. Dimensional variations and allowable chips and cracks are controlled by type.

Table SC-4 summarizes the requirements given in the referenced standards.

Table SC-4: Clay brick and tile requirements

ASTM Specification	Unit	Minimum % solid	Grade		
			Strength	Weight	Type
C34	Load-bearing wall tile	a	yes	yes	no
C56	Non-load-bearing wall tile	b	no	yes	no
C62	Building brick (solid)	75	yes	yes	no
C126	Ceramic glazed units	c	yes	no	yes
C212	Structural facing tile	b	yes	no	yes
C216	Facing brick (solid)	75	yes	yes	yes
C652	Hollow brick	a	yes	yes	yes

Notes:

a. A minimum percent is given in this specification. The percent solid is a function of the requirements for size and/or number of cells as well as the minimum shell and web thicknesses.

b. No minimum percent solid is given in this specification. The percent solid is a function of the requirements for the number of cells and weights per square foot.

c. Solid masonry units minimum percent solid is 75 percent. Hollow masonry units — no minimum percent solid is given in this specification. Their percent solid is a function of the requirements for number of cells and the minimum shell and web thicknesses.

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2.3 C. Provide dimension stone units that conform to ASTM C503, C568, C615, C616, or C629, as specified.

2.3 D. Provide hollow glass units that are partially evacuated and have a minimum average glass face thickness of $^3/_{16}$ in. (4.8 mm). Provide solid glass block units when required. Provide units in which the surfaces intended to be in contact with mortar are treated with polyvinyl butyral coating or latex-based paint. Do not use reclaimed units.

2.3 E. Provide AAC masonry units that conform to ASTM C1691 and ASTM C1693 for the strength class specified in the Contract Documents.

2.3 F. Provide cast stone that conforms to ASTM C1364 as specified.

2.3 G. Provide manufactured stone that conforms to ASTM C1670 as specified.

COMMENTARY

2.3 C. Dimension stone units are typically selected by color and appearance. The referenced standards classify dimension stones by the properties shown in Table SC-5. The values given in the standards serve as minimum requirements. Stone is often ordered from a particular quarry by color rather than the classification method in the standard.

2.3 D. Hollow glass masonry units are formed by fusing two molded halves of glass together to produce a partial vacuum in the resulting space. The resulting glass block units are available in a variety of shapes, sizes, and patterns.

The block edges are usually treated in the factory with a coating that can be clear or opaque. The primary purpose of the coating is to provide an expansion/contraction mechanism to reduce stress cracking and to improve the mortar bond.

2.3 E. AAC masonry units are specified by both compressive strength and density. Various density ranges are given in ASTM C1693 for specific compressive strengths. Generally, the density is specified based on consideration of thermal, acoustical, and weight requirements. AAC masonry is structurally designed based on the minimum compressive strength of the AAC material as determined by ASTM C1693.

Table SC-5: Dimension stone requirements

ASTM Specification	Stone	Absorption	Density	Compressive strength	Modulus of rupture	Abrasion resistance	Acid resistance
C503	Marble	minimum	range	minimum	minimum	minimum	none
C568	Limestone	range	range	range	range	range	none
C615	Granite	minimum	minimum	minimum	minimum	minimum	none
C616	Sandstone	range	range	range	range	range	none
C629	Slate	range	none	none	minimum	minimum	range

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2.4 — Reinforcement, prestressing tendons, and metal accessories

2.4 A. *Steel Reinforcing bars* — Provide deformed steel reinforcing bars that conform to one of the following as specified:

1. ASTM A615/A615M
2. ASTM A706/A706M
3. ASTM A767/A767M
4. ASTM A775/A775M
5. ASTM A996/A996M

COMMENTARY

2.4 — Reinforcement, prestressing tendons, and metal accessories

2.4 A. *Steel Reinforcing bars* — Code Sections 9.1.9.3.1, 9.1.9.3.2, and 11.1.8.6 limit the specified and actual yield strengths of reinforcement used to resist in-plane flexural tension, flexural tension perpendicular to bed joints, in-plane shear, or flexural tension parallel to bed joints in strength design. Test reports should be reviewed to verify conformance with the Code requirement.

See Table SC-6 for a summary of properties.

Table SC-6: Reinforcement and metal accessories

ASTM Specification	Material	Use	Yield strength, ksi (MPa)
A36/A36M	Structural steel	Connectors	36 (250)
A1064/1064M	Steel wire	Joint reinforcement, ties	70 (485)
A240, A480, A580, A666	Stainless steel	Bolts, joint reinforcement, anchors, ties	Varies
A1064/1064M	Steel welded wire reinforcement	Welded wire mesh anchors	70 (485)
A307	Carbon steel	Connectors	Not defined
A1064/1064M	Deformed steel wire	Reinforcement	75 (515)
A1064/1064M	Welded deformed wire reinforcement	Reinforcement	70 (485)
A615/A615M	Carbon-steel	Reinforcement	40, 60 (280, 420)
A996/A996M	Rail and axle steel	Reinforcement	40, 50, 60 (280, 350, 420)
A706/A706M	Low-alloy steel	Reinforcement	60 (420)

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2.4 B. *Prestressing tendons*

1. Provide prestressing tendons that conform to one of the following standards, except for those permitted in Articles 2.4 B.2 and 2.4 B.3:

   a. Wire .... ASTM A421/A421M

   b. Low-relaxation wire........... ASTM A421/A421M

   c. Strand.................................. ASTM A416/A416M

   d. Low-relaxation strand......... ASTM A416/A416M

   e. Bar .... ASTM A722/A722M

2. Wire, strands, and bars not specifically listed in ASTM A416/A416M, A421/A421M, or A722/A722M are permitted, provided that they conform to the minimum requirements in ASTM A416/A416M, A421/A421M, or A722/A722M and are approved by the Architect/Engineer.

3. Bars and wires of less than 150 ksi (1034 MPa) tensile strength and conforming to ASTM A510/A510M, A615/A615M, A706/A706M, A996/A996M, or A1064/A1064M are permitted to be used as prestressed tendons, provided that the stress relaxation properties have been assessed by tests according to ASTM E328 for the maximum permissible stress in the tendon.

2.4 C. *GFRP Reinforcing bars* — Provide solid deformed GFRP reinforcing bars that conform to ASTM D7957/D7957M.

2.4 D. *Joint reinforcement* — Provide joint reinforcement in accordance with the following:

1. ~~that c~~Conforms to ASTM A951 or is fabricated in accordance with ASTM A951 with AISI Type 304 or Type 316 stainless steel wire conforming to ASTM A580/A580M, having a minimum yield strength of 45 ksi (310 MPa) and a minimum ultimate tensile strength of 90 ksi (620 MPa).

2. ~~with m~~Maximum wire size shall not exceed one-half the specified mortar joint thickness. Do not use joint reinforcement with stacked wires whose total thickness exceeds one-half the specified mortar joint thickness.

3. Maximum spacing of cross wires in ladder-type joint reinforcement and of points of connection of cross wires to longitudinal wires of truss-type joint reinforcement shall be 16 in. (400 mm).

~~Exception: Joint reinforcement may be fabricated with AISI Type 304 or Type 316 stainless steel wire conforming~~

COMMENTARY

2.4 B. *Prestressing tendons* — The constructibility aspects of prestressed masonry favor the use of rods or rigid strands with mechanical anchorage in ungrouted construction. Mild strength steel bars have been used in prestressed masonry installations in the United States (Schultz and Scolforo (1991)). The stress-relaxation characteristics of mild strength bars (of less than 150 ksi [1034 MPa]) should be determined by tests and those results should be documented.

2.4 C. *GFRP Reinforcing bars* — The grade of the GFRP bar is not specified because, unlike steel reinforcing bars, GFRP bars are not available in multiple grades. The ultimate tensile strength capacity of the GFRP material varies by manufacturer. ASTM D7957 specifies a minimum guaranteed ultimate tensile force capacity for each size of bar.

2.4 D. *Joint reinforcement* — Code Section 9.1.9.3.2 limits the specified yield strength of joint reinforcement used to resist in-plane shear and flexural tension parallel to bed joints in strength design.

Where vertical reinforcement is present in a masonry wall, diagonal wires in the truss type joint reinforcement will conflict with placement of the vertical reinforcement. Mortar droppings on the diagonal cross wires also make quality grouting more difficult. Consequently, truss-type joint reinforcement should not be specified when the masonry contains vertical reinforcement.

Some manufacturers fabricate joint reinforcement with cross wires spaced at less than 16 in. (400 mm) on center. Joint reinforcement with non-modular dimensioned cross wires can interfere with placement of vertical reinforcement.

2.4 D.1 Commonly available ASTM A580/A580M stainless steel wire does not conform to the minimum yield and tensile strengths required by ASTM A951. ~~The exception allows the use of~~ Joint

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~~to ASTM A580/A580M and having a minimum yield strength of 45 ksi (310 MPa) and a minimum ultimate tensile strength of 90 ksi (620 MPa).~~

2.4 E. *Veneer wire reinforcement* — Provide veneer wire reinforcement that conforms to one of the following:

1. Wire with deformations knurled in conformance with ASTM A951. Wire shall be one of the following types:
   (a) ASTM A1064/A1064M wire meeting the minimum mechanical properties of ASTM A951.
   (b) ASTM A580/A580M, AISI Type 304 or Type 316 stainless steel and having a minimum yield strength of 45 ksi (310 MPa) and a minimum ultimate tensile strength of 90 ksi (620 MPa).
2. Deformed wire that conforms to ASTM A1064/A1064M.
3. Joint reinforcement that conforms to Article 2.4 D.

2.4 ~~E~~F. *Deformed reinforcing wire* — Provide deformed reinforcing wire that conforms to ASTM A1064/A1064M or ASTM A1022/A1022M.

2.4 ~~F~~G. *Welded deformed wire reinforcement* — Provide deformed welded wire reinforcement that conforms to ASTM A1064/A1064M

2.4 H. *Mechanical splices* — Provide mechanical splices that have been demonstrated to develop in tension or compression at least 125 percent of the specified yield strength of the reinforcement. Where indicated, provide mechanical splices that have been demonstrated to develop the specified tensile strength of the reinforcement. Mechanical splices shall be certified for compatibility with the type of reinforcement being spliced.

2.4 ~~G~~I. *Anchors, ties, and accessories* — Provide anchors, ties, and accessories that conform to the following specifications, except as otherwise specified:

1. Plate and bent-bar anchors........ASTM A36/A 36M
2. Sheet-metal anchors and ties .... ASTM A1008/A1008M
3. Welded wire mesh anchors ASTM A1064/A1064M
   a. Minimum wire size W0.3 (MW2.01) at 1/2-in. (12.7 mm) spacing or minimum wire size W0.06 (MW0.03) at 1/4-in. (6.4 mm) spacing.

COMMENTARY

reinforcement fabricated with this wire ~~and requires~~ is permitted, provided that it meets the minimum strength requirements for Type 304 or Type 316 cold-finished wire and is welded and knurled in accordance with ASTM A951.

2.4 E. *Veneer wire reinforcement* — Joint reinforcement is most commonly used as veneer wire reinforcement when masonry backing is used, and typically has three longitudinal wires - one for each face of the backing, and one for the veneer. The cross wires act as veneer ties.

2.4 H. *Mechanical splices* — The strength of mechanical splices is typically demonstrated through testing performed by an independent certification agency. Mechanical splices developing 125 percent of the specified yield strength are commonly referred to as Type 1 splices whereas mechanical splices developing the specified tensile strength are commonly referred to as Type 2 splices. The deformations of deformed bars and deformed wire are different; a splice developed for one type of reinforcement may not develop the intended capacity when used with the other type of reinforcement.

2.4 ~~G~~I.3. Mesh anchors are required to be welded components rather than woven because woven mesh unravels when the pieces are cut to length, making this type of mesh undesirable for the intended use.

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b. Minimum width 2 in. (50.8 mm) less than the masonry nominal thickness and minimum length twice the width.

4. Wire Ties .... ASTM A1064/A1064M

5. Adjustable wall and veneer ties – Provide adjustable wall and veneer ties with at least two pintle legs of wire size W2.8 (MW18) and two horizontal legs of size W2.8 (MW18) embedded into the masonry wythe. Ensure that the maximum clearance between connecting parts of the wall and veneer tie does not exceed 1/16 in. (1.6 mm).

6. Panel anchors (for glass unit masonry) — Provide 1 3/4-in. (44.5-mm) wide, 24-in. (610-mm) long, 20-gage steel strips, punched with three staggered rows of elongated holes, galvanized after fabrication.

2.4 ~~H~~J. *Anchor bolts* — Provide anchor bolts that conform to one of the following as specified:

1. Bent-bar anchor bolts .... ASTM A36/A36M or ASTM F1554
2. Headed anchor bolts .... ASTM A307, Grade A or ASTM F1554

2.4 ~~I~~K. *Stainless steel* — Stainless steel items shall be AISI Type 304 or Type 316, and shall conform to the following:

1. ~~Joint reinforcement .... ASTM A580/A580M~~

~~2.~~ Plate and bent-bar anchors....................................... ASTM A480/A480M and ASTM A666

~~3~~2. Sheet-metal anchors and ties .... ASTM A480/A480M and ASTM A240/A240M

~~4~~3. Wire ties ASTM A580/A580M

COMMENTARY

2.4 ~~G~~I.4. Wire may be used for adjustable and non-adjustable wire ties for multiwythe construction, and for unit wire ties in anchored veneers. Table 13.2.2.5 in the Code also requires that wire components of adjustable ties comply with the requirements for unit wire ties.

2.4 ~~I~~K. *Stainless steel* — Corrosion resistance of stainless steel is greater than that of the other steels listed. Thus, it does not have to be coated for corrosion resistance.

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2.4 ~~J~~L. *Coatings for corrosion protection* — Unless otherwise required, protect carbon steel joint reinforcement, deformed wire placed in mortar, ties, anchors, and steel plates and bars from corrosion by galvanizing or epoxy coating in conformance with the following minimums:

1. Galvanized coatings:
   a. Mill galvanized coatings:
      1) Joint reinforcement and deformed wire .... ASTM A641/A641M (0.1 oz/ft^2) (0.031 kg/m^2)
      2) Sheet-metal ties and sheet-metal anchors .... ASTM A653/A653M Coating Designation G60
   b. Hot-dip galvanized coatings:
      1) Joint reinforcement, deformed wire, ... ASTM A153/A153M (1.50 oz/ft^2) (458 g/m^2)
      2) Sheet-metal ties and sheet-metal anchors .... ASTM A153/A153M Class B
      3) Steel plates and bars (as applicable to size and form indicated) .... ASTM A123/A123M ....or ASTM A153/A153M, Class B
2. Epoxy coatings:
   a. Joint reinforcement, and deformed wire, ...ASTM A884/A884M Class A ....Type 1 — 7 mils (175 µm)
   b. Wire ties... ASTM A899/A899M Class C — 20 mils (508 µm)
   c. Sheet-metal ties and anchors............................... .... 20 mils (508 µm) per surface or manufacturer's specification.

2.4 ~~K~~M. *Corrosion protection for tendons* — Protect tendons from corrosion when they are in exterior walls exposed to earth or weather or walls exposed to a mean relative humidity exceeding 75 percent. Select corrosion protection methods for bonded and unbonded tendons from one of the following:

1. *Bonded tendons* — Encapsulate bonded tendons in corrosion resistant and watertight corrugated ducts complying with Article 2.4 ~~K~~M.1.a. Fill ducts with prestressing grout complying with Article 2.4~~.~~ M.1.b.
   a. Ducts — High-density polyethylene or polypropylene.
      1) Use ducts that are mortar-tight and non-reactive with masonry, tendons, and grout.

COMMENTARY

2.4 ~~J.~~L. *Coatings for corrosion protection* — Amount of galvanizing required increases with severity of exposure (Grimm (1985); Catani (1985); NCMA TEK 12-4D (2006)). Project documents should specify the level of corrosion protection as required by Code Section 6.1.5 and 6.2.1.

2.4 ~~K~~M. *Corrosion protection for tendons* — The specified methods of corrosion protection for unbonded prestressing tendons are consistent with corrosion protection requirements developed for single-strand prestressing tendons in concrete (PTI (2006)). Masonry cover is not sufficient corrosion protection for bonded prestressing tendons in an environment with relative humidity over 75%. Therefore, complete encapsulation into plastic ducts is required. This requirement is consistent with corrosion protection for unbonded tendons. Alternative methods of corrosion protection, such as the use of stainless steel tendons or galvanized tendons, are permitted. Evidence should be provided that the galvanizing used on the tendons does not cause hydrogen embrittlement of the prestressing tendon.

Protection of prestressing tendons against corrosion is provided by a number of measures. Typically, a proprietary system is used that includes sheathing the prestressing tendon with a waterproof plastic tape or duct. Discussion of

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2) Provide ducts with an inside diameter at least 1/4 in. (6.4 mm) larger than the tendon diameter.

3) Maintain ducts free of water if members to be grouted are exposed to temperatures below freezing prior to grouting.

4) Provide openings at both ends of ducts for grout injection.

b. Prestressing grout

1) Select proportions of materials for prestressing grout using either of the following methods as accepted by the Architect/Engineer:

a) Results of tests on fresh and hardened prestressing grout — prior to beginning grouting operations, or

b) Prior documented experience with similar materials and equipment and under comparable field conditions.

2) Use portland cement conforming to ASTM C150, Type I, II, or III, that corresponds to the type upon which selection of prestressing grout was based.

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the various corrosion-protection systems used for prestressed masonry is available in the literature (Garrity (1995)). One example of a corrosion- protection system for the prestressing tendon is shown in Figure SC-5.

Chlorides, fluorides, sulfites, nitrates, or other chemicals in the prestressing grout may harm prestressing tendons and should not be used in harmful concentrations.

Historically, aggregates have not been used in grouts for bonded, post-tensioned concrete construction.

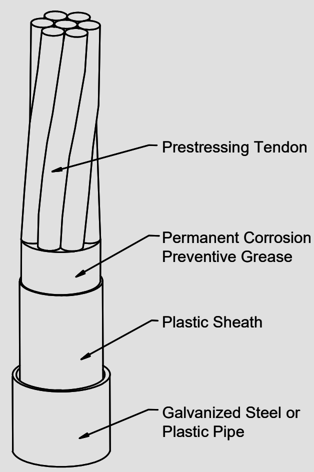

Figure SC-5 — An example of a corrosion-protection system for an unbonded tendon

b. Prestressing grout is a cementitious mixture, not conforming to ASTM C476, and is unique to bonded tendons.

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2.4 ~~K~~M.1.b (Continued)

3) Use the minimum water content necessary for proper pumping of prestressing grout; however, limit the water-cement ratio to a maximum of 0.45 by weight.

4) Discard prestressing grout that has begun to set due to delayed use.

5) Do not use admixtures, unless acceptable to the Architect/Engineer.

6) Use water that is potable and free of materials known to be harmful to masonry materials and reinforcement.

2. *Unbonded tendons* — Coat unbonded tendons with a material complying with Article 2.4 ~~K~~M.2b and wrap with a sheathing complying with Article 2.4 ~~K~~M.2a. Acceptable materials include a corrosion-inhibiting coating material with a tendon sheathing.

a. Provide continuous tendon sheathing over the entire tendon length to prevent loss of coating materials during tendon installation and stressing procedures. Provide a sheathing of medium-density or high-density polyethylene or polypropylene with the following properties:

1) Sufficient strength to withstand damage during fabrication, transport, installation, and tensioning.

2) Water-tightness over the entire sheathing length.

3) Chemical stability without embrittlement or softening over the anticipated exposure temperature range and service life of the structure.

4) Non-reactive with masonry and the tendon corrosion-inhibiting coating.

5) In normal (non-corrosive) environments, a sheathing thickness of at least 0.025 in. (0.6 mm). In corrosive environments, a sheathing thickness of at least 0.040 in. (1.0 mm).

6) An inside diameter at least 0.010 in. (0.3 mm) greater than the maximum diameter of the tendon.

7) For applications in corrosive environments, connect the sheathing to intermediate and fixed anchorages in a watertight fashion, thus providing a complete encapsulation of the tendon.

b. Provide a corrosion-inhibiting coating material with the following properties:

1) Lubrication between the tendon and the sheathing

2) Resist flow from the sheathing within the anticipated temperature range of exposure.

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2.4 ~~K~~M.2.b. *Unbonded tendons* (Continued)

3) A continuous non-brittle film at the lowest anticipated temperature of exposure.
4) Chemically stable and non-reactive with the tendon, sheathing material, and masonry.
5) An organic coating with appropriate polar-moisture displacing and corrosion-preventive additives.
6) A minimum weight not less than 2.5 lb of coating material per 100 ft (37.2 g of coating material per m) of 0.5-in. (12.7-mm) diameter tendon and 3.0 lb of coating material per 100 ft (44.6 g of coating material per m) of 0.6-in. (15.2-mm) diameter tendon. Use a sufficient amount of coating material to ensure filling of the annular space between tendon and sheathing.
7) Extend the coating over the entire tendon length.
8) Provide test results in accordance with Table 5 for the corrosion-inhibiting coating material.

3. Alternative methods of corrosion protection that provide a protection level equivalent to Articles 2.4 ~~K~~M.1 and 2.4 ~~K~~M.2 are permitted. Stainless steel prestressing tendons or tendons galvanized according to ASTM A153/A153M, Class B, are acceptable alternative methods. If galvanized, further evidence must be provided that the coating will not produce hydrogen embrittlement of the steel.

2.4 ~~L~~N. *Prestressing anchorages and couplers*— Provide anchorages and couplers that develop at least 95 percent of the specified breaking strength of the tendons or prestressing steel when tested in an unbonded condition, without exceeding anticipated set.

2.4 ~~M~~O. *Lath* — Provide lath that conforms to one of the following as specified:

1. Expanded metal lath complying with ASTM C847 having a minimum weight of 2.5 lb/yd^2 (1.4 kg/m^2).
2. Woven wire mesh complying with ASTM C1032 having a minimum weight of 1.4 lb/yd^2 (0.76 kg/m^2).
3. Welded wire mesh complying with ASTM C933 having a minimum weight of 1.14 lb/yd^2 (0.618 kg/m^2).
4. Non-metallic lath complying with ASTM C1788.

COMMENTARY

2.4 ~~L~~N. *Prestressing anchorages and couplers* — Typical anchorage and coupling devices are shown in Figure SC-6. Strength of anchorage and coupling devices should be provided by the manufacturer.

Table 5: Performance specification for corrosion-inhibiting coating

Test	Test Method	Acceptance Criteria
Dropping Point, °F (°C)	ASTM D566 or ASTM D2265	Minimum 300 (148.9)
Oil Separation @ 160°F (71.1°C) % by weight	FTMS 791B Method 321.2	Maximum 0.5
Water, % maximum	ASTM D95	0.1
Flash Point, °F (°C) (Refers to oil component)	ASTM D92	Minimum 300 (148.9)
Corrosion Test 5% Salt Fog @ 100°F (37.8°C) 5 mils (0.13 mm), minimum hours (Q Panel type S)	ASTM B117	For normal environments: Rust Grade 7 or better after 720 hr of exposure according to ASTM D610. For corrosive environments : Rust Grade 7 or better after 1000 hr of exposure according to ASTM D610.[1]
Water Soluble Ions[2] a. Chlorides, ppm maximum b. Nitrates, ppm maximum c. Sulfides, ppm maximum	ASTM D512	10 10 10
Soak Test 5% Salt Fog at 100°F (37.8°C) 5 mils (0.13 mm) coating, Q panels, type S. Immerse panels 50% in a 5% salt solution and expose to salt fog	ASTM B117 (Modified)	No emulsification of the coating after 720 hr of exposure
Compatibility with Sheathing a. Hardness and volume change of polymer after exposure to grease, 40 days @ 150°F (65.6°C). b. Tensile strength change of polymer after exposure to grease, 40 days @ 150°F (65.6°C).	ASTM D4289 ASTM D638	Permissible change in hardness 15% Permissible change in volume 10% Permissible change in tensile strength 30%

1 Extension of exposure time to 1000 hours for greases used in corrosive environments requires use of more or better corrosion-inhibiting additives.

2 Procedure: The inside (bottom and sides) of a 33.8 oz (1L) Pyrex beaker, approximate O.D. 4.1 in. (105 mm), height 5.7 in. (145 mm), is thoroughly coated with 35.3 ± 3.5 oz (1000 ± 100 g) corrosion-inhibiting coating material. The coated beaker is filled with approximately 30.4 oz (900 cc) of distilled water and heated in an oven at a controlled temperature of 100°F ± 2°F (37.8°C ± 1°C) for 4 hours. The water extraction is tested by the noted test procedures for the appropriate water soluble ions. Results are reported as ppm in the extracted water.

TMS 602 SPECIFICATION

2.5 — Accessories

2.5 A. Unless otherwise required, provide contraction (shrinkage) joint material that conforms to one of the following standards:

1. ASTM D2000, M2AA-805 Rubber shear keys with a minimum durometer hardness of 80.
2. ASTM D2287, Type PVC 654-4 PVC shear keys with a minimum durometer hardness of 85.
3. ASTM C920.

2.5 B. Unless otherwise required, provide expansion joint material that conforms to one of the following standards:

1. ASTM C920.
2. ASTM D994.
3. ASTM D1056, Class 2A

2.5 C. *Asphalt emulsion* — Provide asphalt emulsion as follows:

1. Metal surfaces.................... ASTM D1187, Type II
2. Porous surfaces... ASTM D1227, Type III, Class 1

2.5 D. *Masonry cleaner*

1. Use potable water and detergents to clean masonry unless otherwise acceptable.
2. Unless otherwise required, do not use acid or caustic solutions.

2.5 E. *Joint fillers* — Use the size and shape of joint fillers specified.

2.5 F. *Veneer tie fasteners* — Provide veneer tie fasteners that conform to the following, unless otherwise specified:

1. Fasteners to wood: AWC NDS.
2. Fasteners to cold-formed metal: AISI S240.
3. Fasteners to concrete: ACI 318.

2.5 G. *Cementitious backer units* — Provide cementitious backer units for veneer systems that comply with ASTM C1325.

COMMENTARY

2.5 — Accessories

2.5 A. and B. Movement joints are used to allow dimensional changes in masonry, minimize random wall cracks, and other distress. Contraction joints (also called control joints or shrinkage joints) are used in concrete masonry to accommodate shrinkage. These joints are free to open as shrinkage occurs. Expansion joints permit clay brick masonry to expand. Material used in expansion joints must be compressible.

Placement of movement joints is recommended by several publications (Grimm (1988); BIA TN 19 (~~2006~~2019); BIA TN 18A (~~2006~~2019); NCMA TEK 10-~~2C~~2D (~~2010~~2019)). Typical movement joints are illustrated in Figure SC-7. Shear keys keep the wall sections on either side of the movement joint from moving out of plane. Proper configuration must be available to fit properly.

ASTM C920 addresses elastomeric joint sealants, either single or multi-component. Sealants that qualify as Grade NS, Class 50 (50% movement capability) or alternatively Class 25 (25% movement capability), Use M are applicable to masonry construction. Expansion joint fillers must be compressible so the anticipated expansion of the masonry can occur without imposing stress.

2.5 D. *Masonry cleaner* — Adverse reactions can occur between certain cleaning agents and masonry units. Hydrochloric acid has been observed to cause corrosion of metal ties. Care should be exercised in its use to minimize this potential problem. Manganese staining, efflorescence, "burning" of the units, white scum removal of the cement paste from the surface of the joints, and damage to metals can occur through improper cleaning. The manufacturers of the masonry units should be consulted for recommended cleaning agents.

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TMS 602 SPECIFICATION

2.5 H. *Lath fasteners* — Provide lath fasteners that comply with ASTM C1063.

2.5 I. *Weep screed* — Provide weep screeds that comply with ASTM C1063.

COMMENTARY

COMMENTARY

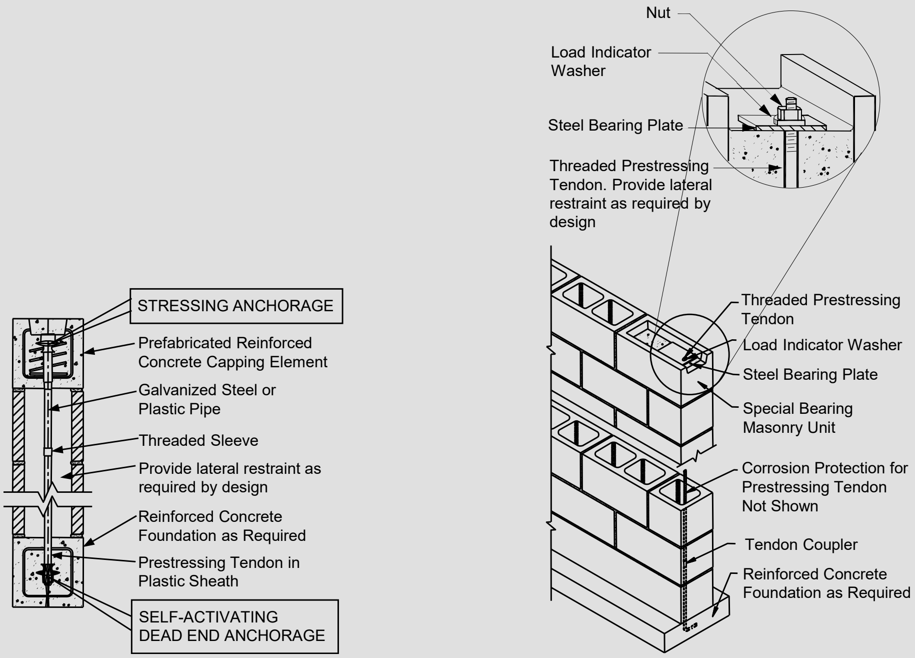

Figure SC-6 — Typical anchorage and coupling devices for prestressed masonry (shown on left using strand tendons and on right using steel bar tendons)

COMMENTARY

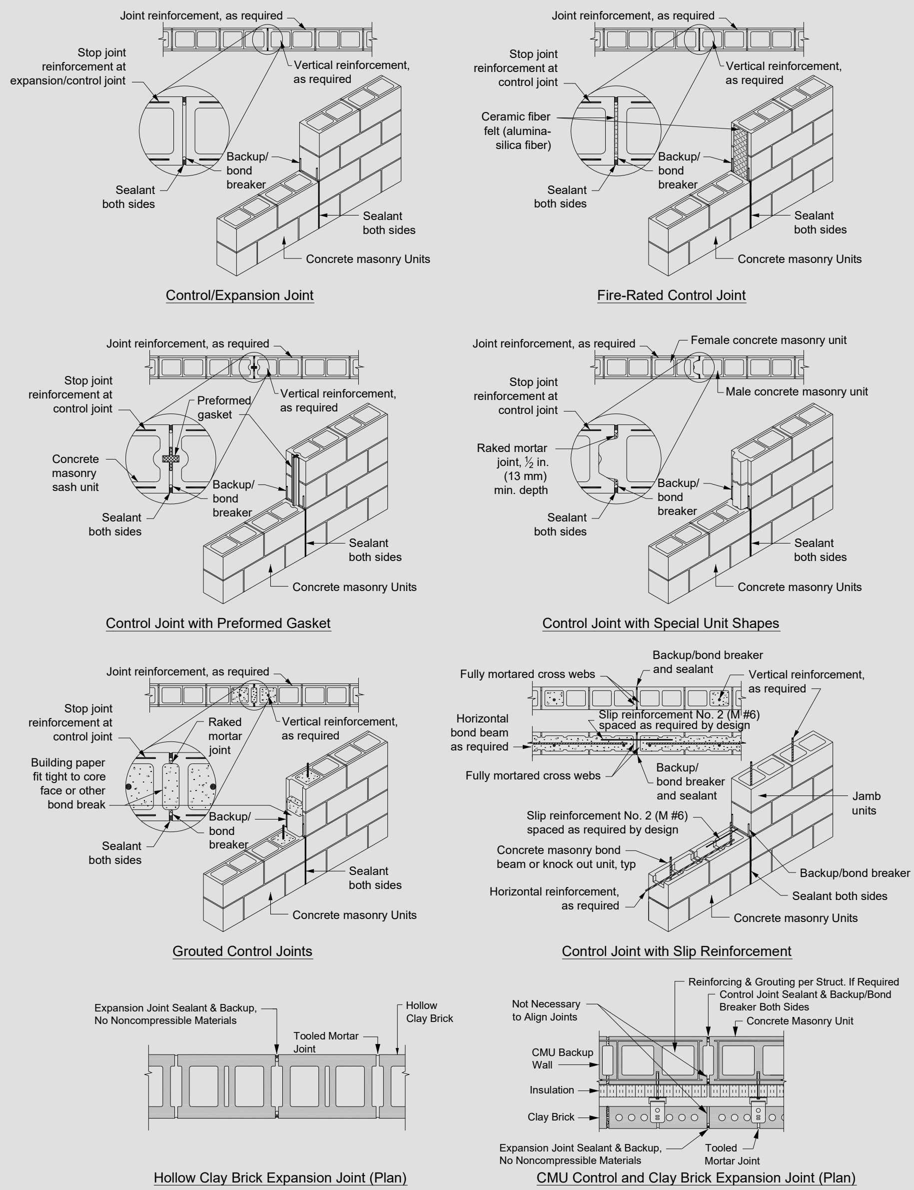

Figure SC-7 — Movement joints

TMS 602 SPECIFICATION

2.6 — Mixing

2.6 A. *Mortar*

1. Mix cementitious materials and aggregates between 3 and 5 minutes in a mechanical batch mixer with a sufficient amount of water to produce a workable consistency. Unless acceptable to the Architect/Engineer, do not hand mix mortar. Maintain workability of mortar by remixing or retempering. Discard mortar which has begun to stiffen or is not used within $2^1/_2$ hr after initial mixing.
2. Limit the weight of mineral oxide or carbon black pigments added to project-site prepared mortar to the following maximum percentages by weight of cement:
   a. Pigmented portland cement-lime mortar
      1) Mineral oxide pigment 10 percent
      2) Carbon black pigment 2 percent
   b. Pigmented mortar cement mortar
      1) Mineral oxide pigment 5 percent
      2) Carbon black pigment 1 percent
   c. Pigmented masonry cement mortar
      1) Mineral oxide pigment 5 percent
      2) Carbon black pigment 1 percent

      Do not add mineral oxide or carbon black pigment to preblended colored mortar or colored cement without the approval of the Architect/Engineer.
3. Do not use admixtures containing more than 0.2 percent chloride ions.
4. *Glass unit masonry* — Reduce the amount of water to account for the lack of absorption. Do not retemper mortar after initial set. Discard unused mortar within $1^1/_2$ hr after initial mixing.

COMMENTARY

2.6 — Mixing

2.6 A. *Mortar* — Caution must be exercised when adding color pigment in field-prepared mortar so that the proportions comply with the Specification requirements.

Preblended products are typically certified to the applicable ASTM Standard and the addition of color at the project site may impact mortar performance.

TMS 602 SPECIFICATION

2.6 B. *Grout*

1. Except for self-consolidating grout, mix grout in accordance with the requirements of ASTM C476.
2. Unless otherwise required, mix grout other than self-consolidating grout to a consistency that has a slump between 8 and 11 in. (203 and 279 mm).
3. Proportioning of self-consolidating grout at the project site is not permitted. Do not add water at the project site except in accordance with the self-consolidating grout manufacturer's recommendations.

COMMENTARY

2.6 B. *Grout* — The two types of grout are fine grout and coarse grout, which are defined by aggregate size. ASTM C476 requires the grout type to be specified by proportion or strength requirements, but not by both methods. ASTM proportion requirements are given in Table SC-7.

The permitted ranges in the required proportions of fine and coarse aggregates are intended to accommodate variations in aggregate type and gradation. As noted in Table 7, the selection of the grout type depends on the size of the space to be grouted. Fine grout is selected for grout spaces with restricted openings. Coarse grout specified under ASTM C476 has a maximum aggregate size that will pass through a ½ in. (12.7 mm) and at least 85% that will pass through a $^3/_8$ in. (9.5 mm) opening.

Grout meeting the proportion specifications of ASTM C476 typically has compressive strength ranges shown in Table SC-8 when measured by ASTM C1019. Grout compressive strength is influenced by the water cement ratio, aggregate content, and the type of units used.

Because grout is placed in an absorptive form made of masonry units, a high water content is required. A slump of at least 8 in. (203 mm) provides a mix fluid enough to be properly placed and supplies sufficient water to satisfy the water demand of the masonry units.

Small cavities or cells require grout with a higher slump than larger cavities or cells. As the surface area and unit shell thickness in contact with the grout decrease in relation to the volume of the grout, the slump of the grout should be reduced. Segregation of materials should not occur.

The grout in place will have a lower water-cement ratio than when mixed. This concept of high slump and absorptive forms is different from that of concrete.

Proportioning of self-consolidating grout at the project site is not permitted because the mixes can be sensitive to variations in proportions, and tighter quality control on the mix is required than can be achieved in the field. Typically, self-consolidating grout comes ready mixed from the manufacturer. Self-consolidating grout may also be available as a preblended dry mix requiring the addition of water at the project site. Manufacturers provide instructions on proper mixing techniques and amount of water to be added. Slump values for self-consolidating grout are expressed as a slump flow because they exceed the 8 in. to 11 in. (203 to 279 mm) slump range for non-self-consolidating grouts.

COMMENTARY

Table SC-7: Grout proportions by volume

Grout type	Cement	Lime	Aggregate damp, loose[1]	
			Fine	Coarse
Fine	1	0 to 1/10	2¼ to 3	—
Coarse	1	0 to 1/10	2¼ to 3	1 to 2

[1] Times the sum of the volumes of the cementitious materials

Table SC-8: Grout strengths

Grout type	Location	Compressive strength, psi (MPa)			Reference
		Low	Mean	High	
Coarse	Lab	1,965 (13.55)	3,106 (21.41)	4,000 (27.58)	ACI-SEASC (1982)
Coarse	Lab	3,611 (24.90)	4,145 (28.58)	4,510 (31.10)	Li and Neis (1986)
Coarse	Lab	5,060 (34.89)	5,455 (37.61)	5,940 (40.96)	ATL (1982)

TMS 602 SPECIFICATION

2.6 C. *Thin-bed mortar for AAC* – Mix thin-bed mortar for AAC masonry as specified by the thin-bed mortar manufacturer.

2.7 — Fabrication

2.7 A. *Steel bar and deformed wire reinforcement*

1. Fabricate standard hooks in accordance with Table 6.
2. Fabricate deformed wire of at least size D11 and bars with minimum inside bend diameter in accordance with Table 6.
3. Fabricate deformed wire larger than size D6 and smaller than size D11 with minimum inside bend diameter of not less than $4d_b$.
4. Fabricate deformed wire up to and including size D6 with minimum inside bend diameter of not less than $2d_b$.
5. Fabricate reinforcing bars in accordance with the fabricating tolerances of ACI 117.
6. Unless otherwise required, bend bars and deformed wire cold and do not heat bars and deformed wire.

COMMENTARY

2.7 — Fabrication

2.7 A. *Steel bar and deformed wire reinforcement* — Standard bends in reinforcing bars and deformed wires are described in terms of the inside diameter of bend because this is easier to measure than the radius of bend. A broad survey of bending practices, a study of ASTM bend test requirements, and a pilot study of and experience with bending Grade 60 (Grade 420) bars were considered in establishing the minimum diameter of bend. The primary consideration was feasibility of bending without breakage. Experience has since established that these minimum bend diameters are satisfactory for general use without detrimental crushing of grout.

The minimum 4 bar diameters bend for the bar sizes commonly used for stirrups and lateral ties is based on accepted industry practice in the United States. The minimum 5 bar diameters bend for certain Grade 40 bars is smaller than that required by ACI 318, but is justified by research (Soric and Tulin, 1987).

The minimum bend diameter requirement for deformed wire smaller than size D11 is taken from ACI 318. ACI's commentary states, in part, "Minimum bend diameters permitted are in most cases the same as those required in the ASTM bend test for wire (ASTM A1064 and A1022)."

TMS 602 SPECIFICATION

2.7 B. *GFRP Reinforcement*

1. Fabricate standard hooks with an extension of 12d_b.
2. Fabricate bars with a minimum inside bend diameter of 6 d_b.
3. Unless otherwise required, do not field bend or straighten GFRP bars.

2.7 C. *Prefabricated masonry*

1. Unless otherwise required, provide prefabricated masonry that conforms to the provisions of ASTM C901.
2. Unless otherwise required, provide prefabricated masonry lintels that have an appearance similar to the masonry units used in the wall surrounding each lintel.
3. Mark prefabricated masonry for proper location and orientation.

COMMENTARY

ACI 117 Specification~~s~~ for Tolerances for Concrete Construction and Materials (117-10) and Commentary contains fabrication tolerances for steel reinforcement. Recommended methods and standards for preparing design drawings, typical details, and drawings for the fabrications and placing of reinforcing steel in reinforced concrete structures are given in ACI 315R (~~1999~~2018) and may be used as a reference in masonry design and construction.

2.7 B. *GFRP Reinforcement* — The standard hook extension of 12d_b is recommended by ACI 440.1R (2015). The minimum inside bend diameter is based on ASTM D7957/D7957M.

2.7 C. *Prefabricated masonry* — ASTM C901 addresses the requirements for prefabricated masonry panels, including materials, structural design, dimensions and variations, workmanship, quality control, identification, shop drawings, and handling.

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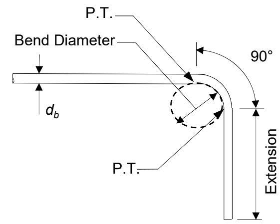
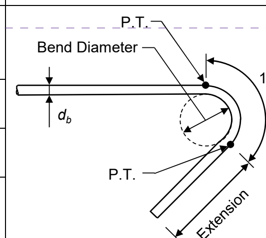
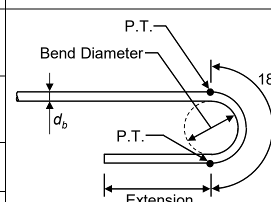
Table 6: Standard Hooks Geometry and Minimum Inside Bend Diameters for Steel Reinforcing Bars and Deformed Wire

Standard Hook Type and Use	Steel Grade	Reinforcement Size	Minimum Inside Bend Diameter	Minimum Extension	Standard Hook Figures
90 Degree Hook – Reinforcement	40 (M280)	No.3 - No. 7 (M#10 - #22)	$5d_b$	12 d_b	 P.T. = Point of Tangency
	50 or 60 (M350 or 420)	No. 3 - No. 8 (M#10 - #25)	6 d_b	12 d_b	
	50 or 60 (M350 or 420)	No. 9 - No. 11 (M#29 - #36)	8 d_b	12 d_b	
	75	D11 – D31 (MD 71 – MD 200)	6 d_b	12 d_b	
90 Degree Hook – Stirrups & Lateral Ties	40, 50, 60 (M280,350 or 420)	No.3 - No.5 (M#10 - #16)	$4d_b$	$6d_b$ but not less than 3 in. (76 mm)	
	40 (M280)	No.6 and No.7 (M#19 - #22)	5 d_b	6 d_b	
	50 or 60 (M350 or 420)	No.6 - No.8 (M#19 - #25)	6 d_b	6 d_b	
	75	D11 – D31 (MD 71 – MD 200)	4 d_b	$6d_b$ but not less than 3 in. (76 mm)	
135 Degree Hook – Stirrups & Lateral Ties	40, 50, 60 (M280, 350 or 420)	No.3 - No.5 (M#10 - #16)	4 d_b	$6d_b$ but not less than 3 in. (76 mm)	
	40 (M280)	No.6 and No.7 (M#19 - #22)	5 d_b	6 d_b	
	50 or 60 (M350 or 420)	No.6 - No.8 (M#19 - #25)	6 d_b	6 d_b	
	75	D11 – D31 (MD 71 – MD 200)	4 d_b	$6d_b$ but not less than 3 in. (76 mm)	
180 Degree Hook – Reinforcement	40 (M280)	No.3 - No.7 (M#10 - #22)	5 d_b	$4d_b$ but not less than 2-1/2 in. (64 mm)	
	50 or 60 (M350 or 420)	No.3 - No.8 (M#10 - #25)	6 d_b	$4d_b$ but not less than 2-1/2 in. (64 mm)	
	50 or 60 (M350 or 420)	No.9 - No.11 (M#29 -#36)	8 d_b	4 d_b	
	75	D11 – D31 (MD 71 – MD 200)	6 d_b	$4d_b$ but not less than 2 1/2 in. (64 mm)	
180 Degree Hook – Stirrups & Lateral Ties	40, 50, 60 (M280, 350 or 420)	No.3 - No.5 (M#10 - #16)	4 d_b	$4d_b$ but not less than 2-1/2 in. (64 mm)	
	40 (M280)	No.6 and No.7 (M#19 - #22)	5 d_b	4 d_b	
	50 or 60 (M350 or 420)	No.6 - No.8 (M#19 - #25)	6 d_b	4 d_b	
	75	D11 – D31 (MD 71 – MD 200)	4 d_b	$4d_b$ but not less than 2 1/2 in. (64 mm)	

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PART 3 — EXECUTION

TMS 602 SPECIFICATION

3.1 — Inspection

3.1 A. Prior to the start of masonry construction, the Contractor shall verify:

1. That foundations are constructed within a level alignment tolerance of $\pm^1/_2$ in. (12.7 mm).
2. That reinforcing dowels are positioned in accordance with the Project Drawings.

3.1 B. If stated conditions are not met, notify the Architect/Engineer.

3.2 — Preparation

3.2 A. Clean reinforcement and shanks of anchor bolts by removing mud, oil, or other materials that will adversely affect or reduce bond at the time mortar or grout is placed. Reinforcement with rust, mill scale, or both are acceptable without cleaning or brushing provided that the dimensions and weights, including heights of deformations, of a cleaned sample are not less than required by the ASTM specification that governs this reinforcement.

3.2 B. *Foundation preparation* — Prior to placing masonry, remove laitance, loose aggregate, and anything else that would prevent mortar from bonding to the foundation. When required, roughen the base surface.

3.2 C. *Wetting masonry units*

1. *Concrete masonry* — Unless otherwise required, do not wet concrete masonry or AAC masonry units before laying. Wet cutting is permitted.
2. *Clay or shale masonry* — Wet clay or shale masonry units having initial absorption rates in excess of 1 g per min. per in.^{2} (0.0016 g per min. per mm^2), when measured in accordance with ASTM C67, so the initial rate of absorption will not exceed 1 g per min. per in.^{2} (0.0016 g per min. per mm^2) when the units are used. Lay wetted units when surface dry. Do not wet clay or shale masonry units having an initial absorption rate less than 0.2 g per min. per in.^{2} (0.00031 g per min. per mm^2).

3.2 D. *Debris* — Construct grout spaces free of mortar droppings, debris, loose aggregates, and any material deleterious to masonry grout.

3.2 E. *Reinforcement* — Place reinforcement and lateral ties in grout spaces prior to grouting.

COMMENTARY

3.1 — Inspection

3.1 A. The tolerances in this Article are taken from ACI 117 (1990). The dimensional tolerances of the supporting concrete are important because they control such aspects as mortar joint thickness and bearing area dimensions, which influence the performance of the masonry. The specified width of the foundation is obviously more critical than its specified length. A foundation wider than specified will not normally cause structural problems.

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3.2 C. *Wetting masonry units* — Concrete masonry units increase in volume when wetted and shrink upon subsequent drying. Water introduced during wet cutting is localized and does not significantly affect the shrinkage potential of concrete masonry. Clay masonry units with high absorption rates dry the mortar/unit interface. This may result in a lower extent of bond between the units and mortar, which may create paths for moisture intrusion. Selection of compatible units and mortar can mitigate this effect.

3.2 D. *Debris* — Continuity in the grout is critical for uniform stress distribution. A reasonably clean space to receive the grout is necessary for this continuity. Cells need not be vacuumed to achieve substantial cleanliness. Inspection of the bottom of the space prior to grouting is critical to ensure that it is substantially clean and does not have accumulations of deleterious materials that would prevent continuity of the grout.

3.2 E. *Reinforcement* — Loss of bond and misalignment of the reinforcement can occur if it is not placed prior to grouting.

TMS 602 SPECIFICATION

3.2 F. *Cleanouts* — Provide cleanouts in the bottom course of masonry for each grout pour when the grout pour height exceeds 5 ft 4 in. (1.63 m).

1. Construct cleanouts so that the space to be grouted can be cleaned and inspected. In solid grouted masonry, space cleanouts horizontally a maximum of 32 in. (813 mm) on center.
2. Construct cleanouts with an opening of sufficient size to permit removal of debris. The minimum opening dimension shall be 3 in. (76.2 mm).
3. After cleaning, close cleanouts with closures braced to resist grout pressure.

3.3 — Masonry erection

3.3 A. *Bond pattern* — Unless otherwise required, lay masonry in running bond.

3.3 B. *Placing mortar and units*

1. *Bed joints at foundations* — In the starting course on foundations and other supporting members, construct bed joints so that the bed joint thickness is at least ¼ in. (6.4 mm) and not more than:

   a. ¾ in. (19.1 mm) when the masonry is ungrouted or partially grouted.

   b. 1¼ in. (31.8 mm) when the first course of masonry is solid grouted and supported by a concrete foundation.

COMMENTARY

3.2 F. *Cleanouts* — Cleanouts can be constructed by removing the exposed face shell of units in hollow unit grouted masonry or individual units when grouting between wythes. The purpose of cleanouts is to allow the grout space to be adequately cleaned prior to grouting. They can also be used to verify reinforcement placement and tying.

3.3 B. *Placing mortar and units* — Article 3.3 B applies to masonry construction in which the units support their own weight.

1. *Bed joints at foundations* — The range of permitted mortar bed joint thickness at foundations for solid grouted masonry walls is compatible with the foundation tolerances of Article 3.1 A.1. Figure SC-8 shows the allowable foundation tolerance of ± ½ in. and the relationship of the mortar bed joint. The contractor should coordinate the mortar bed joint at foundations with the coursing requirements so that the intended masonry module is met at critical points, such as story height and top of wall, window and door openings. Either fine or coarse grout for the first course of masonry may be placed when normal masonry grouting is performed for fully grouted masonry, or may be placed after the first course is laid and prior to placement of additional courses when the masonry is not fully grouted.

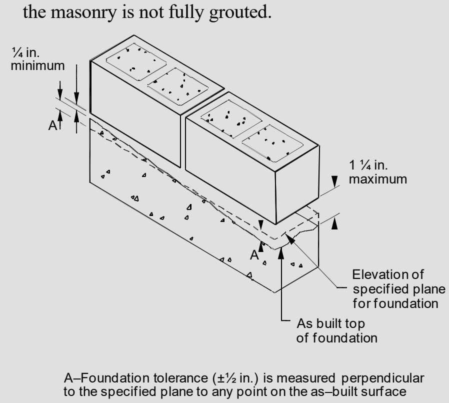

Figure SC-8 — Mortar bed joint thickness for solid grouted walls on a foundation

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TMS 602 SPECIFICATION

3.3 B. *Placing mortar and units* (Continued)

2. *Bed and head joints* — Unless otherwise required, construct $^3/_8$-in. (9.5-mm) thick bed and head joints, except at foundation or with glass unit masonry. Provide glass unit masonry bed and head joint thicknesses in accordance with Article 3.3 B.6.c. Provide AAC masonry bed and head joint thicknesses in accordance with Article 3.3 B.8.b. Construct joints that also conform to the following:

   a. Fill holes not specified in exposed and below grade masonry with mortar.

   b. Unless otherwise required, tool joint with a round jointer when the mortar is thumbprint hard.

   c. Remove masonry protrusions extending $^1/_2$ in. (12.7 mm) or more into cells or cavities to be grouted.

3. *Hollow units* — Place hollow units so:

   a. Face shells of bed joints are fully mortared.

   b. Webs are fully mortared in:

      1) all courses of columns and pilasters;

      2) when necessary to confine grout or insulation.

   c. Head joints are mortared, a minimum distance from each face equal to the face shell thickness of the unit.

   d. Vertical cells to be grouted are aligned and unobstructed openings for grout are provided in accordance with the Project Drawings.

COMMENTARY

b. Tooling mortar joints enhances bond to the masonry units.

c. Mortar protrusions are limited to projections of less than 1/2 inch (12.7 mm) into the cell or cavity so that the grout flow is not unduly impeded. There is no benefit in removing conforming mortar protrusions because fluid grout surrounds the mortar protrusion and provides a mechanical interlock between the masonry and grout.

3. *Hollow units* — Face shell mortar bedding of hollow units is standard, except in locations detailed in Article 3.3 B.3.b. Figure SC-9 shows the typical placement of mortar for hollow-unit masonry walls. In partially grouted walls, however, cross webs next to cells that are to be grouted are usually mortared. If full mortar beds throughout are required for structural capacity, for example, the specifier must so stipulate in the Project Specifications or Project Drawings.

Normal construction practices can create mortar joints with minor imperfections and small voids that have no significant effect on the masonry assembly. Mortar joints on the face of masonry need to be filled to provide the specified mortar joint finish.

TMS 602 SPECIFICATION

3.3 B. *Placing mortar and units* (Continued)

4. *Solid units* — Unless otherwise required, place mortar so that bed and head joints are fully mortared and:
   a. Do not fill head joints by slushing with mortar.
   b. Construct head joints by shoving mortar tight against the adjoining unit.
   c. Do not deeply furrow bed joints.
5. *Open-end units with beveled ends* — Fully grout open-end units with beveled ends. Head joints of open-end units with beveled ends need not be mortared. At the beveled ends, form a grout key that permits grout within 5/8 in. (15.9 mm) of the face of the unit. Tightly butt the units to prevent leakage of grout.
6. *Glass units*
   a. Apply a complete coat of asphalt emulsion, not exceeding $^{1}/_{8}$ in. (3.2 mm) in thickness, to panel bases.
   b. Lay units so head and bed joints are filled solidly. Do not furrow mortar.
   c. Unless otherwise required, construct head and bed joints of glass unit masonry ¼ in. (6.4 mm) thick, except that vertical joint thickness of radial panels shall not be less than $^{1}/_{8}$ in. (3.2 mm). The bed-joint thickness tolerance shall be minus $^{1}/_{16}$ in. (1.6 mm) and plus $^{1}/_{8}$ in. (3.2 mm). The head-joint thickness tolerance shall be plus or minus $^{1}/_{8}$ in. (3.2 mm).
   d. Do not cut glass units.

COMMENTARY

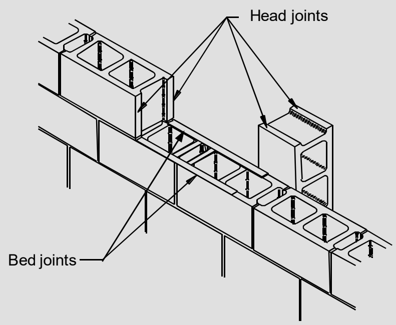

Figure SC-9 — Mortar placement of hollow units in walls

4. *Solid units* — Normal construction practices can create mortar joints with minor imperfections and small voids that have no significant effect on the masonry assembly. Mortar joints on the face of masonry need to be filled to provide the specified mortar joint finish.

TMS 602 SPECIFICATION

3.3 B. *Placing mortar and units* (Continued)

7. *All units*

   a. Place clean units while the mortar is soft and plastic. Remove and re-lay in fresh mortar any unit disturbed to the extent that initial bond is broken after initial positioning.

   b. Except for glass units, cut exposed edges or faces of masonry units smooth, or position so that exposed faces or edges are unaltered manufactured surfaces.

   c. When the bearing of a masonry wythe on its support is less than two-thirds of the wythe thickness, notify the Architect/Engineer.

8. *AAC masonry*

   a. Place mortar for leveling bed joint in accordance with the requirements of Article 3.3 B.1.

   b. Lay subsequent courses using thin-bed mortar. Use special notched trowels manufactured for use with thin-bed mortar to spread thin-bed mortar so that it completely fills the bed joints. Unless otherwise specified in the Contract Documents, similarly fill the head joints. Spread mortar and place the next unit before the mortar dries. Place each AAC unit as close to head joint as possible before lowering the block onto the bed joint. Avoid excessive movement along bed joint. Make adjustments while thin-bed mortar is still soft and plastic by tapping to plumb and bring units into alignment. Set units into final position, in mortar joints at least 1/16-in. (1.5-mm) thick, by striking on the end and top with a rubber mallet.

   c. Lay units in alignment with the plane of the wall. Align vertically and plumb using the first course for reference. Make minor adjustments by sanding the exposed faces of the units and the bed joint surface with a sanding board manufactured for use with AAC masonry.

COMMENTARY

8 *AAC masonry* — AAC masonry can be cut, shaped and drilled with tools that are capable of cutting wood; however, saws, sanding boards, and rasps manufactured for use with AAC are recommended for field use. Because thin-bed mortar joints do not readily allow for plumbing of a wall, the ability of AAC masonry to be easily cut and shaped allows for field adjustment to attain required tolerances. Workmanship requirements for AAC masonry are provided in ASTM C1692.

TMS 602 SPECIFICATION

3.3 C. *Installing collar joints, cavities and drainage spaces*

1. Collar joints — Unless otherwise required, solidly fill collar joints less than 3/4 in. (19.1 mm) wide with mortar as the project progresses.
2. Cavities and drainage spaces – Build masonry with cavities and drainage spaces as indicated on the Project Drawings and as specified.

3.3 D. *Installing adhered veneer*

1. For veneer applied to lath and scratch coat, install lath in accordance with ASTM C1063 and install scratch coat in accordance with ASTM C926. Use self-furring lath or fur away from backup surfaces by at least 1/4 in. (6.3 mm). On light frame backing, install lath and scratch coat over sheathing.
2. For veneer applied directly to masonry, concrete, or cement backer units, the surface shall be free of coatings, debris, membranes, or similar materials that would inhibit bond to the backing.
3. For veneer applied to cement backer units, install cement backer units and treat joints between cement backer units as specified.
4. For all installations:
   a. Unless otherwise required, install weep screed.
   b. For polymer modified mortar setting beds, apply mortar per requirements of appropriate ANSI A118.4 or A118.15.
   c. Install units with tight-fit joints or mortar joints in accordance with the Contract Documents. Install the veneer unit, filling the space between the veneer unit and the backing with mortar. Use sufficient setting bed mortar and installation pressure to create a slight excess of mortar to be forced out between the edges of the veneer units.
   d. Unless otherwise required, install mortar between units and tool when thumbprint hard.

3.3 E. *Embedded items and accessories* — Install embedded items and accessories as follows:

1. Construct chases as masonry units are laid.
2. Install pipes and conduits passing horizontally through masonry partitions.
3. Place pipes and conduits passing horizontally through pilasters or columns.
4. Place horizontal pipes and conduits in and parallel to plane of walls.
5. Install and secure connectors, flashing, weep holes, weep vents, nailing blocks, and other accessories.

COMMENTARY

3.3 C *Installing collar joints, cavities and drainage spaces* — The drainage space may contain materials such as mortar droppings, mortar protrusions, drainage media, veneer ties, and mortar dropping collection devices provided that moisture is able to drain from the space.

3.3 D *Installing adhered veneer* — Article 3.3 D applies to adhered veneer in which the backing supports the weight of the units. Additional information on the installation of adhered veneer is in ASTM C1780 (~~2017~~2020), BIA (2014) and ~~MVMA~~ NCMA (~~2017~~2021).

Proprietary systems or products may have requirements that are different than the generic prescriptive requirements shown here.

~~**3.3 D 4.b.** Proprietary systems or products may have setting bed thickness that are different than the generic prescriptive requirements shown here.~~

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TMS 602 SPECIFICATION

3.3 E. Members (Continued)

6. Install movement joints.
7. Aluminum — Do not embed aluminum conduits, pipes, and accessories in masonry, grout, or mortar, unless they are effectively coated or isolated to prevent chemical reaction between aluminum and cement or electrolytic action between aluminum and steel.

3.3 F. *Bracing of masonry* — Design, provide, and install bracing that will assure stability of masonry during construction.

3.3 G. *Site tolerances* — Erect masonry within the following tolerances from the specified dimensions.

1. Dimensional tolerances
   a. In cross section or elevation
      ....$-^1/_4$ in. (6.4 mm), $+^1/_2$ in. (12.7 mm)
   b. Mortar joint thickness
      bed joints between masonry courses
      ....$\pm^1/_8$ in. (3.2 mm)
      bed joint between flashing and masonry
      .... - ½ in. (12.7 mm), $+^1/_8$ in. (3.2 mm)
      head...........- $^1/_4$ in. (6.4 mm), + $^3/_8$ in. (9.5 mm)
      collar ....$-^1/_4$ in. (6.4 mm), + $^3/_8$ in. (9.5 mm)
      glass unit masonry............see Article 3.3 B.6.c
      AAC thin-bed mortar joint thickness ....
      .... -0, + $^1/_8$ in. (3.2 mm)
   c. Collar joint, cavity width, or drainage space, except for masonry walls passing frame construction
      ....$-^1/_4$ in. (6.4 mm), + $^3/_8$ in. (9.5 mm)

COMMENTARY

3.3 F. *Bracing of masonry* — Construction stability of masonry walls may be accomplished by bracing masonry walls (externally and/or internally), or other appropriate methods. Reinforced grouted masonry walls are more stable than ungrouted walls during the construction phase. Requirements for bracing masonry walls for worker safety during construction is also required by OSHA 29 CFR 1926.706 (OSHA (1994)). For guidance on bracing of masonry walls for wind, consult Standard Practice for Bracing Masonry Walls Under Construction (MCAA (2012)).

3.3 G. *Site tolerances* — Tolerances are established to limit eccentricity of applied load. Because masonry is usually used as an exposed material, it is subjected to tighter dimensional tolerances than those for structural frames. These tolerances set the standard for quality of workmanship and are based on structural performance, not aesthetics.

Mortar is required to bond masonry courses, but it is not required when masonry is laid on top of flashing.

The provisions for cavity width shown are for the space between wythes of non-composite masonry or between the veneer and backing of veneer assemblies. The provisions do not apply to situations where masonry extends past floor slabs, spandrel beams, or other structural members.

TMS 602 SPECIFICATION

3.3 G (Continued)

2. Members

a. Variation from level:
bed joints
.... $\pm^1/_4$ in. (6.4 mm) in 10 ft (3.05 m)
.... $\pm^1/_2$ in. (12.7 mm) maximum

top surface of load-bearing walls
.... $\pm^1/_4$ in. (6.4 mm) in 10 ft (3.05 m)
$\pm^1/_2$ in. (12.7 mm) maximum

b. Variation from plumb
.... $\pm^1/_4$ in. (6.4 mm) in 10 ft (3.05 m)
.... $\pm^3/_8$ in. (9.5 mm) in 20 ft (6.10 m)
.... $\pm^1/_2$ in. (12.7 mm) maximum

c. True to a line
.... $\pm^1/_4$ in. (6.4 mm) in 10 ft (3.05 m)
.... $\pm^3/_8$ in. (9.5 mm) in 20 ft (6.10 m)
.... $\pm^1/_2$ in. (12.7 mm) maximum

d. Alignment of columns and walls
(bottom versus top)
....$\pm^1/_2$ in. (12.7 mm) for load-bearing walls and columns
.$\pm^3/_4$ in. (19.1 mm) for non-load-bearing walls

3. Location of members

a. Indicated in plan
.... $\pm^1/_2$ in. (12.7 mm) in 20 ft (6.10 m)
.... $\pm^3/_4$ in. (19.1 mm) maximum

b. Indicated in elevation
.... $\pm^1/_4$ in. (6.4 mm) in story height
.... $\pm^3/_4$ in. (19.1 mm) maximum

4. If the above conditions cannot be met due to previous construction, notify the Architect/ Engineer.

COMMENTARY

Tolerances for variation from level are shown in Figure SC-10

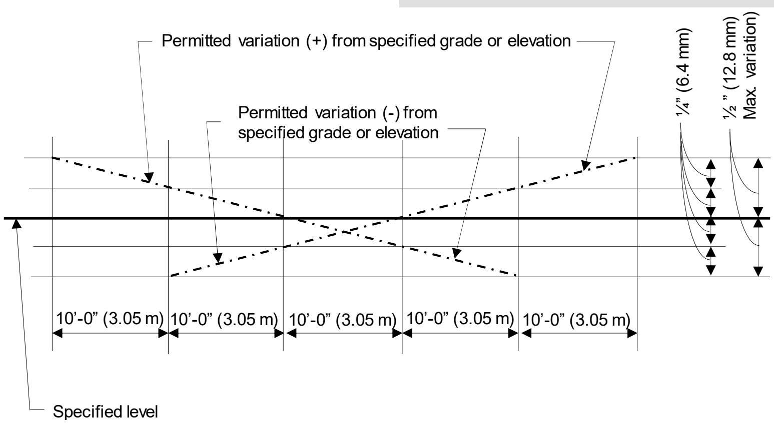


Figure SC-10 — Tolerance for variation from level

TMS 602 SPECIFICATION

3.4 — Reinforcement, tie, and anchor installation

3.4 A. *Basic requirements* — Place reinforcement, ties, and anchors in accordance with the sizes, types, and locations indicated on the Project Drawings and as specified. Do not place dissimilar metals in contact with each other.

3.4 B. *Reinforcement*

1. Support reinforcement to prevent displacement caused by construction loads or by placement of grout or mortar, beyond the allowable tolerances.

2. Completely embed reinforcing bars and deformed wires larger than one-half the mortar joint thickness in grout in accordance with Article 3.5, except as permitted by Article 3.4.B.6

3. Maintain clear distance between reinforcing bars or mechanical splices and the interior of masonry unit or formed surface of at least $^{1}/_{4}$ in. (6.4 mm) for fine grout and $^{1}/_{2}$ in. (12.7 mm) for coarse grout, except where cross webs of hollow units are used as supports for horizontal reinforcement. Maintain the same clear distance when deformed wire is specified to be embedded in grout.

4. Place reinforcing bars~~, and~~ deformed wire, and mechanical splices in grout maintaining the following minimum cover:

   a. Masonry face exposed to earth or weather: 2 in. (50.8 mm) for bars larger than No. 5 (M #16) and mechanical splices; 1½ in. (38.1 mm) for deformed wire and No. 5 (M #16) bars or smaller.

   b. Masonry not exposed to earth or weather: 1½ in. (38.1 mm).

5. Maintain minimum clear distance between parallel bars~~, and~~ parallel deformed wires and mechanical splices of the nominal reinforcement size or 1 in. (25.4 mm), whichever is greater. For mechanical splices, the nominal reinforcement size is the greatest cross-sectional dimension of the mechanical splice.

COMMENTARY

3.4 — Reinforcement, tie, and anchor installation

The requirements given ensure that:

a. galvanic action is inhibited,

b. location is as assumed in the design,

c. there is sufficient clearance for grout and mortar to surround reinforcement, ties, and anchors so stresses are properly transferred,

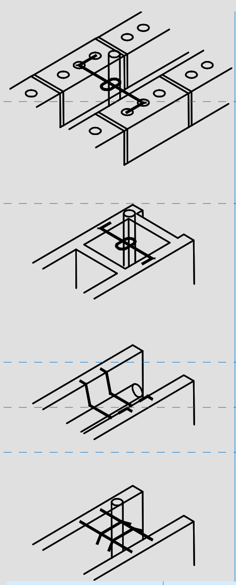
d. corrosion is delayed, and

e. compatible lateral deflection of wythes is achieved.

3.4 B. *Reinforcement*

1. Tolerances for placement of reinforcement in masonry first appeared in the 1985 Uniform Building Code (UBC (1985)). Reinforcement location obviously influences structural performance of the member. Figure SC-11 illustrates several devices used to secure bar reinforcement.

3, 4, and 5. The orientation of mechanical splices that have localized protrusions from the main body of the coupler, such as bolt heads or ports, may need to be adjusted to achieve the specified clearances and cover distances.



While not required by Code, rebar positioners are one way to comply with reinforcement placement requirements.

Figure SC-11 — ~~Typical~~ Examples of positioners for reinforcement

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TMS 602 SPECIFICATION

3.4 B. *Reinforcement* (Continued)

6. In columns and pilasters:

   a. Maintain minimum clear distance between vertical bars or vertical deformed wires of one and one-half times the nominal reinforcement size or 1½ in. (38.1 mm), whichever is greater. Do not substitute bars with deformed wires unless permitted by the Architect/Engineer.

   b. When a lateral tie or combination of ties crosses a web or interior face shell and exceeds the specified thickness of the mortar bed joint, remove part of the unexposed portions of the units to maintain proper clearance for grout around the tie(s).

   c. When a lateral tie or combination of ties crosses a web or interior face shell and does not exceed the specified thickness of the mortar joint, either remove part of the unexposed portions of the units to maintain proper clearance for grout around the tie(s), or fully bed the webs and face shells with mortar including bridging the gaps between adjacent and parallel webs or face shells.

7. Splice reinforcement only where indicated on the Project Drawings, unless otherwise acceptable. When splicing bars with mechanical splices, comply with manufacturer's installation requirements. When splicing bars by welding, provide welds in conformance with the provisions of AWS D 1.4. When splicing wire reinforcement by welding, provide welds as specified. When splicing reinforcement by lapping, provide lap length that meets or exceeds the lap length specified.

8. Unless accepted by the Architect/Engineer, do not bend reinforcement after it is embedded in grout or mortar.

COMMENTARY

6. Figure SC-15 illustrates conditions where lateral ties cross webs and interior face shells and shows removal of portions of the units to permit adequate grout clearance around the ties. When the lateral tie is permitted to be partially embedded in mortar because it does not exceed the thickness of the specified mortar bed joint, corrosion protection is assured by requiring the mortar bed for the tie to extend over the gap between units.

8. The Architect/Engineer has many factors to consider when deciding whether to permit reinforcement to be bent after being embedded in grout or mortar: the effect on mechanical properties of the reinforcement, whether preheating of reinforcement will be required, potential damage to the grout or mortar, and potential damage to the protective coating on the reinforcement (if any). More information on bending of bar reinforcement embedded in concrete may be found in CRSI (2015).

   In addition to deformed bars, this provision applies to joint reinforcement, deformed wire reinforcement, and welded wire reinforcement. These wire materials should not be bent because these cold-formed components have lower ductility than reinforcing bars. Furthermore, these materials are unlikely to lay flat after being bent multiple times and the protective coating is more likely to be compromised.

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TMS 602 SPECIFICATION

3.4 B. *Reinforcement* (Continued)

9. *Noncontact lap splices* — Position bars and deformed wire spliced by noncontact lap splice no farther apart transversely than one-fifth the length of lap nor more than 8 in. (203 mm)

10 *GFRP Reinforcement* — Field cut GFRP reinforcement using cutting methods specified by or acceptable to the Architect/Engineer. Do not use shears or flame cutting to field cut GFRP bars. Seal cut ends if required by the GFRP reinforcing bar manufacturer or Architect/Engineer. Repair surface damage due to cutting if required by the Architect/Engineer.

11. *Joint reinforcement, veneer wire reinforcement, and deformed wire reinforcement in mortar*

a. Place ~~joint~~ reinforcemen~~t and deformed wire~~ so that longitudinal wires are embedded in mortar with a minimum cover of $^{1}/_{2}$ in. (12.7 mm) when not exposed to weather or earth; or $^{5}/_{8}$ in. (15.9 mm) when exposed to weather or earth. Center veneer wire reinforcement on the wythe when the veneer is constructed with solid units.

3.4 B.11 (Continued)

COMMENTARY

When bending of joint reinforcement is not permitted, use of joint reinforcement tees and corners for continuity (as required by Article 3.4 B.11.c) is limited to intersecting walls that are erected at the same time. Joint reinforcement tees and corners protrude significantly from the wall ends and, if used when intersecting walls are constructed independently, will be bent and rebent to avoid a potential safety issue from the protruding reinforcement and to allow construction of the second wall.

9. *Noncontact lap splices* — Lap splices may be constructed with the bars or wires in adjacent grouted cells if the requirements of this article are met. The lap length specified by the Architect/Engineer is a minimum. Often, the as-constructed lap length will have to be longer than the specified minimum to permit noncontact lap splices spaced transversely up to 8 inches (203 mm).

11. *Joint reinforcement, veneer wire reinforcement, and deformed wire reinforcement in mortar*

a. There must be a minimum protective cover for the ~~deformed wire and joint~~ reinforcement as shown in Figure SC-12. Deeply tooled mortar joints, which provide inadequate protective cover, should be avoided.

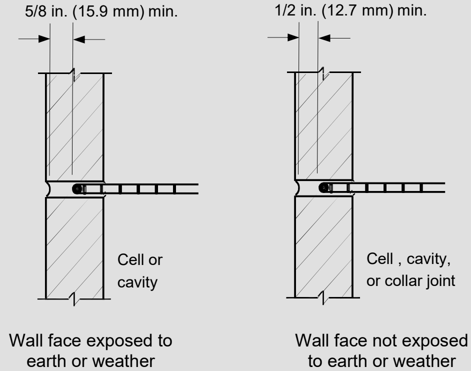

Figure SC-12 Joint Reinforcement Cover Requirements

TMS 602 SPECIFICATION

3.4 B.11 (Continued)

b. Provide minimum 8-in. (203-mm) lap splices for joint reinforcement and veneer wire reinforcement. Provide minimum lap splice length of 48 wire diameters for deformed wire. Do not stack ~~joint~~ reinforcement ~~wires~~ at laps.

c. Provide continuity of ~~joint~~ reinforcement ~~and deformed wire~~ at corners and intersections, unless a movement joint is detailed at the corner or intersection. Do not bend the wires more than once when field fabricating ~~joint~~ reinforcement corners except for minor adjustments to meet as-built conditions.

d. Ensure that all ends of ~~deformed wire and longitudinal wires of joint~~ reinforcement at laps are embedded in mortar or grout.

COMMENTARY

b. The requirement for lap splices provides continuity of the ~~joint~~ reinforcement ~~and deformed wire~~. Wire stacking is prohibited because workmanship quality is adversely affected. Stacked wires fill all or most of the mortar joint thickness (depending on wire size), making it difficult to lay masonry units within the tolerances of Article 3.3G.

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c. Continuity of reinforcement at wall intersections and corners may be provided by installing prefabricated tees and corners. Field fabricated configurations for all reinforcing types may also be acceptable provided the corrosion protection requirements are met. Prefabricated tees and corners of reinforcement protrude significantly from the wall. Bending and rebending ~~these installed components~~prodruding reinforcement during construction to avoid interference should not be permitted because wire integrity and wire flatness are adversely affected. Consequently, only minor adjustments after the initial bend are permitted. Alternatively, ~~joint~~ reinforcement placed in mortar~~and deformed wire~~ could be limited to connecting intersecting walls that are erected at the same time.

d. Where laps ~~in deformed wire~~ occur in ~~longitudinal wires of joint~~ reinforcement the minimum embedment provisions of Article 3.4 B.11.a apply. Figure SC-13 shows typical joint reinforcement lap splices in mortar or grout.

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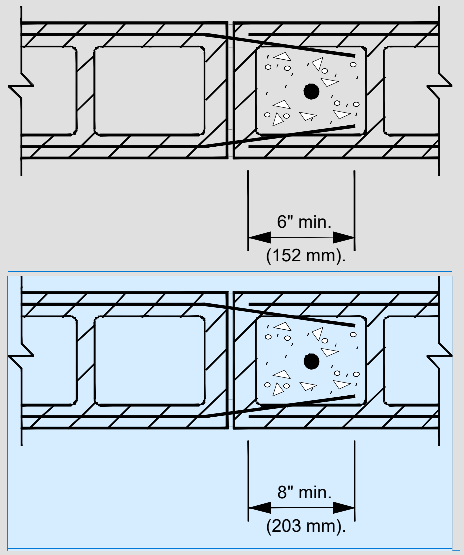
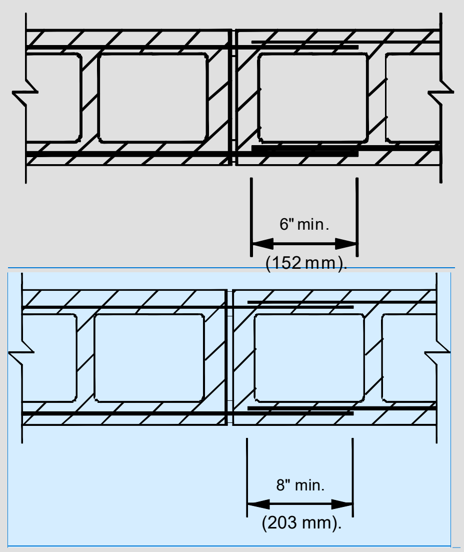
TMS 602 SPECIFICATION

COMMENTARY



Grout embedment



Mortar embedment

Figure SC-13 Joint Reinforcement Lap Splices

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TMS 602 SPECIFICATION

3.4 B. *Reinforcement* (Continued)

12. *Placement tolerances*

a. Place reinforcing bars and deformed wire in walls and flexural members within a tolerance of ± 1/2 in. (12.7 mm) when the distance from the centerline of reinforcement to the opposite face of masonry, *d*, is equal to 8 in. (203 mm) or less, ± 1 in. (25.4 mm) for *d* equal to 24 in. (610 mm) or less but greater than 8 in. (203 mm), and ± 1 1/4 in. (31.8 mm) for *d* greater than 24 in. (610 mm).

b. Place vertical bars and deformed wire within:

1) 2 in. (50.8 mm) of the required location along the length of the wall when the wall segment length exceeds 24 in. (610 mm).

2) 1 in. (25.4 mm) of the required location along the length of the wall when the wall segment length does not exceed 24 in. (610 mm)

c. If it is necessary to move reinforcement more than one reinforcement diameter or a distance exceeding the tolerance stated above to avoid interference with other reinforcing steel, conduits, or embedded items, notify the Architect/Engineer for acceptance of the resulting arrangement of reinforcement.

COMMENTARY

12. *Placement tolerances*

a. Ways to measure *d* distance in various common masonry members are shown in Figures SC-14 through SC-16 (Chrysler (~~2010~~2017)). The maximum permissible tolerance for placement of reinforcement in a wall, beam, and column is based on the *d* dimension of that member.

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In masonry, the *d* dimension is measured perpendicular to the length of the member and is defined in the Specification as the distance from the center of the reinforcement to the compression face of masonry.

In a wall subject to out-of-plane loading, the distance, *d*, to the compression face is normally the larger distance when reinforcement is offset from the center of the wall, as shown in Figure SC-14.

The *d* dimension in masonry columns will establish the maximum allowable tolerance for placement of the vertical reinforcement. As shown in Figure SC-15, two dimensions for each vertical bar or wire must be considered to establish the allowable tolerance for placement of the vertical reinforcement in each primary direction.

The *d* dimension in a masonry beam will establish the maximum allowable tolerance for placement of the horizontal reinforcement within the depth of the beam. As shown in Figure SC-16, the distance to the top of beam is used to establish the allowable tolerance for placement of the reinforcement.

b. The tolerance for placement of vertical reinforcement along the length of the wall is shown in Figure SC-14. As shown, the allowable tolerance is +/- 2 in. (50.8 mm), except for wall segments not exceeding 24 in. (610 mm) where the allowable tolerance is decreased to +/- 1 in. (25.4 mm). This tolerance applies to each reinforcing bar or wire relative to the specified location in the wall. An accumulation of tolerances could result in reinforcement placement that interferes with cross webs in hollow masonry units.

COMMENTARY

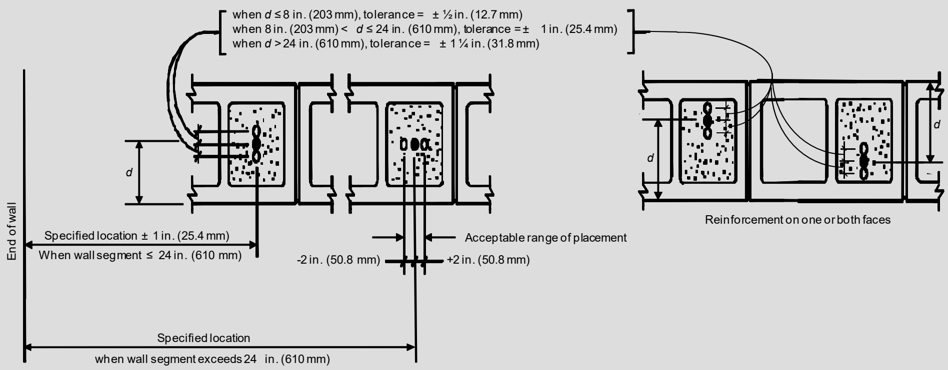

Figure SC-14 — Typical 'd' distance in a wall

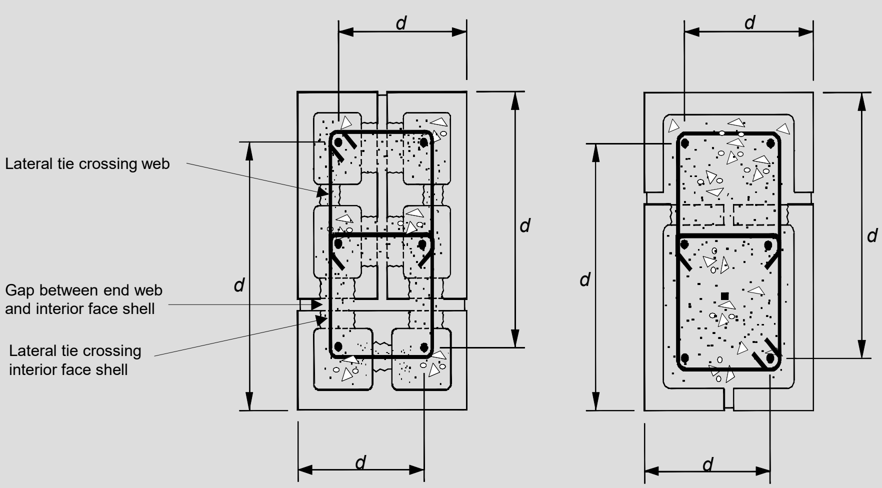

Figure SC-15 — Typical 'd' distance in a column

TMS 602 SPECIFICATION

3.4 B.12. *Reinforcement, Placement tolerances* (Continued)

d. Foundation dowels that interfere with unit webs are permitted to be bent to a maximum of 1 in. (25.4 mm) horizontally for every 6 in. (152 mm) of vertical height.

COMMENTARY

d. Misaligned foundation dowels may interfere with placement of the masonry units. Interfering dowels may be bent in accordance with this provision (see Figure SC-17) (Stecich et al (1984); NCMA TEK 3-2A (2005)). Removing a portion of the web to better accommodate the dowel may also be acceptable as long as the dowel is fully encapsulated in grout and masonry cover is maintained.

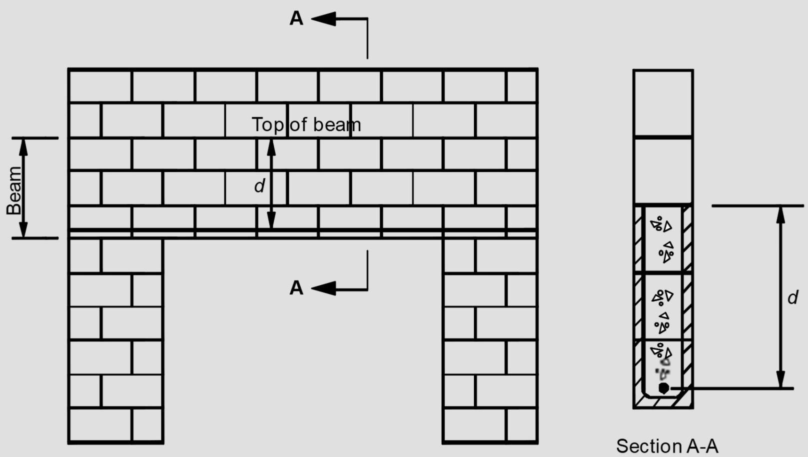

Figure SC-16 — Typical 'd' distance in a beam

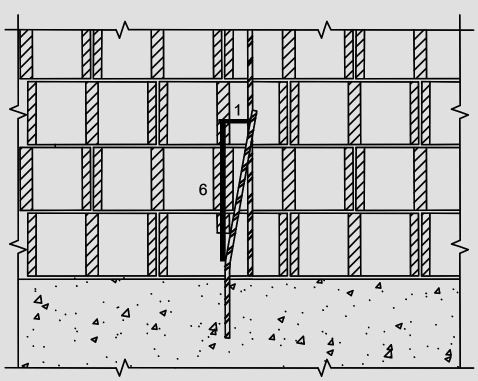

Figure SC-17 — Permitted Bending of Foundation Dowels

TMS 602 SPECIFICATION

3.4 C. *Wall ties*

1. Embed the ends of wall ties in mortar joints. Embed wall tie ends at least $^1/_2$ in. (12.7 mm) into the outer face shell of hollow units. Embed wire wall ties at least $1^1/_2$ in. (38.1 mm) into the mortar bed of solid masonry units or solid grouted hollow units.
   a. Do not use Z-ties in masonry constructed with hollow units.
   b. Do not use ties formed with cavity drips in non-composite masonry.
2. Unless otherwise required, bond wythes not bonded by headers with wall ties as follows:

Wire size	*Minimum number of wall ties required*
W1.7 (MW11)	One per 2.67 ft^2 (0.25 m^2)
W2.8 (MW18)	One per 4.50 ft^2 (0.42 m^2)

   The maximum spacing between ties is 36 in. (914 mm) horizontally and 24 in. (610 mm) vertically.
3. Unless accepted by the Architect/Engineer, do not bend wall ties after being embedded in grout or mortar.
4. Unless otherwise required, install adjustable ties in accordance with the following requirements:
   a. One tie for each 1.77 ft^2 (0.16 m^2) of wall area.
   b. Do not exceed 16 in. (406 mm) horizontal or vertical spacing.
   c. Do not use adjustable wall ties when the misalignment of bed joints from one wythe to the other exceeds $1^1/_4$ in. (31.8 mm).
5. Install wire ties perpendicular to a vertical line on the face of the wythe from which they protrude and perpendicular to the face of the wythe in which they are embedded. Comply with manufacturer's installation requirements, including placement tolerances. Where one-piece ties or joint reinforcement are used, ensure that the bed joints of adjacent wythes are aligned.
6. Unless otherwise required, provide additional unit ties around openings larger than 16 in. (406 mm) in either dimension. Space ties around perimeter of opening at a maximum of 3 ft (0.91 m) on center. Place ties within 12 in. (305 mm) of opening.
7. Unless otherwise required, provide unit ties within 12 in. (305 mm) of unsupported edges at horizontal or vertical spacing given in Article 3.4 C.2.

COMMENTARY

3.4 C. *Wall ties* — TMS 402 does not permit the use of cavity wall ties with drips, nor the use of Z-ties in ungrouted, hollow unit masonry. The requirements for adjustable ties are shown in Figure SC-18. A summary of limitations and applications for Z-ties and cavity drip wall ties are listed in Table SC-9. Other types of wall ties may be available to satisfy the connection requirement of composite and non-composite walls.

Table SC-9: Z and cavity drip wall tie applications[1]

Unit Type	Composite Walls		Non-Composite Walls	
	Z-Ties	Cavity Drip Ties	Z-Ties	Cavity Drip Ties
Hollow	NP	P	NP	NP
Solid	P	P	P	NP

[1] P = Permitted, NP = Not permitted

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COMMENTARY

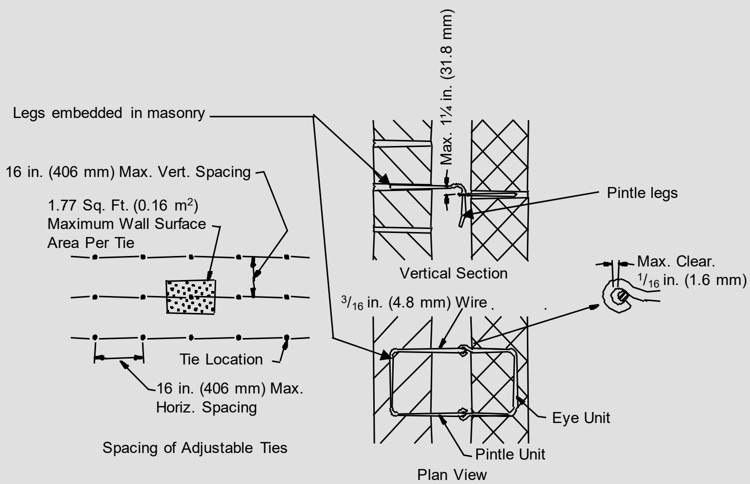

Figure SC-18 — Adjustable ties

TMS 602 SPECIFICATION

3.4 ~~E~~D. *Veneer ties*

1. Place corrugated sheet-metal anchors, sheet-metal unit wire and adjustable ties as follows:

   a. With solid units, embed veneer ties in mortar joint and extend into the ~~veneer~~ masonry wythe a minimum of 1½ in. (38.1 mm), with at least 5/8 in. (15.9 mm) mortar cover to the outside face of the veneer.

   b. With hollow units, embed ~~anchors~~veneer ties in mortar or grout and extend into the ~~veneer~~ masonry wythe a minimum of 1 ½ in. (38.1 mm), with at least 5/8 in. (15.9 mm) mortar or grout cover to outside face.

2. ~~Do not use~~Install adjustable veneer ties such that~~when~~ the vertical offset between the two pieces does not exceed~~s~~ 1-1/4 in. (31.8 mm).

3. Embed longitudinal wires of joint reinforcement in the mortar joint with at least 5/8-in. (15.9-mm) mortar cover on each side.

~~4. Unless otherwise required, provide at least one veneer tie for each 1.78 ft² (0.165 m²) of wall area with a spacing not to exceed 16 in. (406 mm) in either the vertical or horizontal direction.~~

~~5~~4. Unless otherwise required, place veneer ties within 16 in. (406 mm) of supported edges, within 12 in. (305 mm) of unsupported edges, openings, and movement joints, and within 8 in. (203 mm) of the top of the veneer.

~~6~~5. Place veneer ties within a tolerance of ± 1 in. (25.4 mm) of specified location.

~~7~~6. Unless otherwise required, fasten veneer ties to the backing according to the following:

   a. Backing of wood: NDS.

   b. Backing of cold-formed metal: AISI S240.

   c. Backing of concrete: ACI 318.

~~8~~7. Install veneer ~~anchors~~ ties perpendicular to a vertical line on the face of the backup from which they protrude and perpendicular to the face of the veneer. Comply with manufacturer's installation requirements, including placement tolerances.

COMMENTARY

3.4 ~~E~~D. *Veneer ties* — Minimum embedment requirements have been established for each of the anchor types to ensure load resistance against push-through or pullout of the mortar joint.

2. Proper anchorage of veneer ties into veneers using hollow masonry units can be satisfied by mortaring veneer ties in bed joints or on the cross-webs of the units; by grouting the cells or cores adjacent to the veneer tie; or by following the veneer tie manufacturer's requirements for installing the veneer tie into the cell or core above or below the bed joint and filling the cell or core containing the veneer tie with mortar or grout.

~~4. The maximum tie spacing is intended to achieve compliance with TMS 402 requirements for Enhanced Prescriptive Design. The Contract Documents may allow increased tie spacings for projects that qualify for Basic Prescriptive Design or that have been designed using the engineered methods.~~

~~6~~5. Veneer ties that are within tolerance will result in slight variations of the tributary area of the tie. This is considered to be acceptable.

~~**3.4 E.** *Veneer ties (continued)*~~

~~7~~6. Install the number of fasteners as required in the manufacturer's instructions.

Commented [PJS97]: 19-VG-064

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Commented [PJS100]: 19-VG-064

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TMS 602 SPECIFICATION

3.4 ~~D~~E. *Anchor bolts*

1. Embed headed and bent-bar anchor bolts larger than ¼ in. (6.4 mm) diameter in grout that is placed in accordance with Article 3.5 A and Article 3.5 B. Anchor bolts of ¼ in. (6.4 mm) diameter or less are permitted to be placed in grout or mortar bed joints that have a specified thickness of at least ½ in. (12.7 mm) thickness.

2. For anchor bolts placed in the top of grouted cells and bond beams, maintain a clear distance between the bolt and the face of masonry unit of at least ¼ in. (6.4 mm) when using fine grout and at least ½ in. (12.7 mm) when using coarse grout.

3. For anchor bolts placed through the face shell of a hollow masonry unit, drill a hole that is tight-fitting to the bolt or provide minimum clear distance that conforms to Article 3.4 D.2 around the bolt and through the face shell. For the portion of the bolt that is within the grouted cell, maintain a clear distance between the bolt and the face of masonry unit and between the head or bent leg of the bolt and the formed surface of grout of at least ¼ in. (6.4 mm) when using fine grout and at least ½ in. (12.7 mm) when using coarse grout.

4. Place anchor bolts with a clear distance between parallel anchor bolts not less than the nominal diameter of the anchor bolt, nor less than 1 in. (25.4 mm).

COMMENTARY

3. A $^{1}/_{16}$ in. (1.6 mm) maximum annular ring between the anchor bolt and masonry unit will provide for a tight fitting bolt. Quality assurance/control (QA/QC) procedures should assure that there is sufficient clearance around the bolts prior to grout placement. These procedures should also include observation during grout placement to assure that grout completely surrounds the bolts, as required by the QA Tables in Article 1.6 A

   The clear distance requirement for grout to surround an anchor bolt does not apply where the bolt fits tightly in the hole of the face shell, but is required where the bolt is placed in an oversized hole in the face shell and where grout surrounds the anchor bolt in a grouted cell. See Figure SC-19.

COMMENTARY

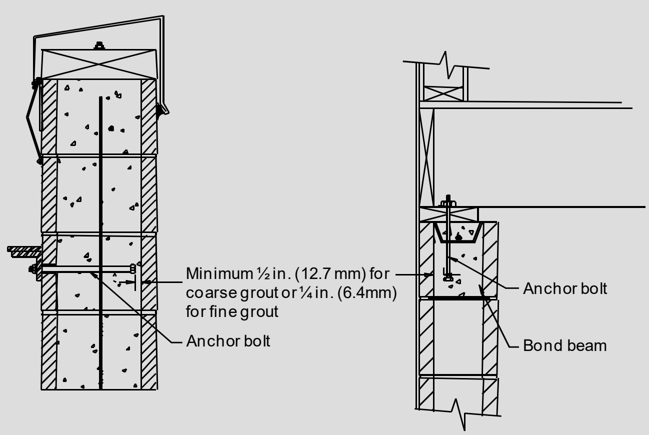

Figure SC-19 — Anchor bolt clearance requirements for headed anchor bolts – bent-bars are similar

Commented [PJS102]: 19-VG-064

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TMS 602 SPECIFICATION

~~3.4 E.~~ ~~*Veneer ties*~~

~~1. Place corrugated sheet-metal anchors, sheet-metal unit wire and adjustable ties as follows:~~

~~a. With solid units, embed veneer ties in mortar joint and extend into the veneer a minimum of 1½ in. (38.1 mm), with at least 5/8 in. (15.9 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 1 ½ in. (38.1 mm), with at least 5/8 in. (15.9 mm) mortar or grout cover to outside face.~~

~~2. Do not use adjustable veneer ties when the vertical offset between the two pieces exceeds 1-1/4 in. (31.8 mm).~~

~~3. Embed longitudinal wires of joint reinforcement in the mortar joint with at least 5/8-in. (15.9-mm) mortar cover on each side.~~

~~4. Unless otherwise required, provide at least one veneer tie for each 1.78 ft² (0.165 m²) of wall area with a spacing not to exceed 16 in. (406 mm) in either the vertical or horizontal direction.~~

~~5. Unless otherwise required, place veneer ties within 16 in. (406 mm) of supported edges, within 12 in. (305 mm) of unsupported edges, openings, and movement joints, and within 8 in. (203 mm) of the top of the veneer.~~

~~6. Place veneer ties within a tolerance of ± 1 in. (25.4 mm) of specified location.~~

~~7. Unless otherwise required, fasten veneer ties to the backing according to the following:~~

~~a. Backing of wood: NDS.~~

~~b. Backing of cold-formed metal: AISI S240.~~

~~c. Backing of concrete: ACI 318.~~

~~8. Install veneer anchors perpendicular to a vertical line on the face of the backup from which they protrude and perpendicular to the face of the veneer. Comply with manufacturer's installation requirements, including placement tolerances.~~

3.4 F. *Adhered veneer fasteners* — Place veneer fasteners in adhered veneer assemblies within a tolerance of ± 0.25 in. (6.4 mm) of specified location.

COMMENTARY

~~**3.4 E.** *Veneer ties* Minimum embedment requirements have been established for each of the anchor types to ensure load resistance against push-through or pullout of the mortar joint.~~

~~2. Proper anchorage of veneer ties into veneers using hollow masonry units can be satisfied by mortaring veneer ties in bed joints or on the cross-webs of the units; by grouting the cells or cores adjacent to the veneer tie; or by following the veneer tie manufacturer's requirements for installing the veneer tie into the cell or core above or below the bed joint and filling the cell or core containing the veneer tie with mortar or grout.~~

~~4. The maximum tie spacing is intended to achieve compliance with TMS 402 requirements for Enhanced Prescriptive Design. The Contract Documents may allow increased tie spacings for projects that qualify for Basic Prescriptive Design or that have been designed using the engineered methods.~~

~~6. Veneer ties that are within tolerance will result in slight variations of the tributary area of the tie. This is considered to be acceptable.~~

~~**3.4 E.** *Veneer ties (continued)*~~

~~7. Install the number of fasteners as required in the manufacturer's instructions.~~

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TMS 602 SPECIFICATION

3.4 G. *Glass unit masonry panel anchors* — When used instead of channel-type restraints, install panel anchors as follows:

1. Unless otherwise required, space panel anchors at 16 in. (406 mm) in both the jambs and across the head.
2. Embed panel anchors a minimum of 12 in. (305 mm), except for panels less than 2 ft (0.61 m) in the direction of embedment. When a panel dimension is less than 2 ft (0.61 m), embed panel anchors in the short direction a minimum of 6 in. (152 mm), unless otherwise required.
3. Provide two fasteners, capable of resisting the required loads, per panel anchor.

3.4 H. *Welded wire mesh anchors* — When connecting masonry partition walls to other walls for lateral support of the partition wall, install welded wire mesh anchors as follows:

1. Locate welded wire mesh anchors at vertical intervals that do not exceed 16 in. (406 mm).
2. Locate welded wire mesh anchors with minimum 5/8 in. (15.9 mm) mortar or grout cover to exterior surfaces and minimum 1/2 in. (13 mm) mortar or grout cover to interior surfaces.
3. While maintaining the required cover, position the welded wire mesh anchor with equal embedment in each wall unless one wall does not have sufficient width to place one half the specified length.
4. Fully grout the cells below and above the welded wire mesh anchor and completely embed the welded wire mesh anchor in mortar or grout.

COMMENTARY

TMS 602 SPECIFICATION

3.5 — Grout placement

3.5 A. *Placing time* — Place grout within $1^1/_2$ hr from introducing water in the mixture and prior to initial set.

1. Discard site-mixed grout that does not meet the specified slump without adding water after initial mixing.
2. For ready-mixed grout:
   a. Addition of water is permitted at the time of discharge to adjust slump.
   b. Discard ready-mixed grout that does not meet the specified slump without adding water, other than the water that was added at the time of discharge.

   The time limitation is waived as long as the ready-mixed grout meets the specified slump.

3.5 B. *Confinement* — Confine grout to the areas indicated on the Project Drawings. Use material to confine grout that permits bond between masonry units and mortar.

3.5 C. *Grout pour height* — Do not exceed the maximum grout pour height given in Table 7.

COMMENTARY

3.5 — Grout placement

Grout may be placed by pumping or pouring from large or small buckets. The amount of grout to be placed and contractor experience influence the choice of placement method.

The requirements of this Article do not apply to prestressing grout.

3.5 A. *Placing time* — Grout placement is often limited to 1½ hours after initial mixing, but this time period may be too long in hot weather (initial set may occur) and may be unduly restrictive in cooler weather. One indicator that the grout has not reached initial set is a stable and reasonable grout temperature. However, sophisticated equipment and experienced personnel are required to determine initial set with absolute certainty.

Article 3.5 A.2 permits water to be added to ready-mixed grout to compensate for evaporation that has occurred prior to discharge. Replacement of evaporated water is not detrimental to ready-mixed grout. However, water may not be added to ready-mixed grout after discharge.

3.5 B. *Confinement* — Certain locations in the wall may not be grouted in order to reduce dead loads or allow placement of other materials such as insulation or wiring. Cross webs adjacent to cells to be grouted can be bedded with mortar to confine the grout. Metal lath, plastic screening, or other items can be used to plug cells below bond beams.

3.5 C. *Grout pour height* — Table 7 in the Specification has been developed as a guide for grouting procedures. The designer can impose more stringent requirements if so desired. The recommended maximum height of grout pour (see Figure SC-20) corresponds with the least clear dimension of the grout space. The minimum width of grout space is used when the grout is placed in collar joints. The minimum cell dimensions are used when grouting cells of hollow masonry units, including consideration of vertical alignment of cells. As the height of the pour increases, the minimum grout space increases. The grout space dimensions are smallest clear dimensions, considering projections or obstructions into the grout space and the diameter of horizontal reinforcement, as illustrated in Figure SC-21. The grout space requirements of Table 7 are based on coarse and fine grouts as defined by ASTM C476, which defines aggregate size, and cleaning practice to permit the complete filling of grout spaces and adequate consolidation using typical methods of construction.

Grout pour heights and minimum dimensions that meet the requirements of Table 7 do not automatically mean that the grout space will be filled.

Grout spaces smaller than specified in Table 7 have been used successfully in some areas. When the contractor asks for acceptance of a grouting procedure that does not meet the limits in Table 7, construction of a grout demonstration panel is required. Destructive or non-destructive evaluation can confirm that filling and adequate consolidation have been achieved. The Architect/Engineer should establish criteria for the grout demonstration panel to assure that critical masonry components included in the construction will be represented in the demonstration panel. Because a single grout demonstration panel erected prior to masonry construction cannot account for all conditions that may be encountered during construction, the Architect/Engineer should establish inspection procedures to verify grout placement during construction. These inspection procedures should include destructive or non-destructive evaluation to confirm that filling and adequate consolidation have been achieved.

Table 7: Grout space requirements

Grout type[1]	Maximum grout pour height, ft (m)	Minimum clear width of grout space,[2,3] in. (mm)	Minimum clear grout space dimensions for grouting cells of hollow units,[3,4] in. x in. (mm x mm)
Fine	1 (0.30)	$^3/_4$ (19.1)	1$^1/_2$ x 2 (38.1 x 50.8)
Fine	5.33 (1.63)	2 (50.8)	2 x 3 (50.8 x 76.2)
Fine	12.67 (3.86)	2$^1/_2$ (63.5)	2$^1/_2$ x 3 (63.5 x 76.2)
Fine	24 (7.32)	3 (76.2)	3 x 3 (76.2 x 76.2)
Coarse	1 (0.30)	1$^1/_2$ (38.1)	1$^1/_2$ x 3 (38.1 x 76.2)
Coarse	5.33 (1.63)	2 (50.8)	2$^1/_2$ x 3 (63.5 x 76.2)
Coarse	12.67 (3.86)	2$^1/_2$ (63.5)	3 x 3 (76.2 x 76.2)
Coarse	24 (7.32)	3 (76.2)	3 x 4 (76.2 x 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 reinforcement from the as-built cross section of the grout space. Select the grout type and maximum grout pour height based on the minimum clear space.

4 Minimum grout space dimension for AAC masonry units shall be 3 in. (76.2 mm) x 3 in. (76.2 mm) or a 3 in. (76.2 mm) diameter cell.

TMS 602 SPECIFICATION

3.5 D. *Grout lift height*

1. For grout conforming to Article 2.2 A:

   a. Where the following conditions are met, place grout in lifts not exceeding 12 ft 8 in. (3.86 m).

      i. The masonry has cured for at least 4 hours.

      ii. The grout slump is maintained between 10 and 11 in. (254 and 279 mm).

      iii. No intermediate reinforced bond beams are placed between the top and the bottom of the pour height.

   b. When the conditions of Articles 3.5 D.1.a.i and 3.5 D.1.a.ii are met but there are intermediate bond beams within the grout pour, limit the grout lift height to the bottom of the lowest bond beam that is more than 5 ft 4 in. (1.63 m) above the bottom of the lift, but do not exceed a grout lift height of 12 ft 8 in. (3.86 m).

   c. When the conditions of Article 3.5 D.1.a.i or Article 3.5 D.1.a.ii are not met, place grout in lifts not exceeding 5 ft 4 in. (1.63 m).

2. For self-consolidating grout conforming to Article 2.2:

   a. When placed in masonry that has cured for at least 4 hours, place in lifts not exceeding the grout pour height.

   b. When placed in masonry that has not cured for at least 4 hours, place in lifts not exceeding 5 ft 4 in. (1.63 m) or the grout pour height, whichever is less.

3.5 E. *Consolidation*

1. Consolidate grout at the time of placement.

   a. Consolidate grout pours 12 in. (305 mm) or less in height by mechanical vibration or by puddling.

   b. Consolidate pours exceeding 12 in. (305 mm) in height by mechanical vibration, and reconsolidate by mechanical vibration after initial water loss and settlement has occurred, but prior to the initial set and loss of plasticity.

2. Consolidation or reconsolidation is not required for self-consolidating grout.

COMMENTARY

3.5 D. *Grout lift height* — A lift is the height to which grout is placed into masonry in one continuous operation (see Figure SC-20). After placement of a grout lift, water is absorbed by the masonry units. Following this water loss, a subsequent lift may be placed on top of the still plastic grout.

Grouted construction develops fluid pressure in the grout space. Grout pours composed of several lifts may develop this fluid pressure for the full pour height. The faces of hollow units with unbraced ends can break out. Wythes may separate. The wire ties between wythes may not be sufficient to prevent this from occurring. Higher lifts may be used with self-consolidating grout because its fluidity and its lower initial water-cement ratio result in reduced potential for fluid pressure problems.

The 4-hour time period is stipulated for grout lifts over 5 ft 4 in. (1.63 m) to provide sufficient curing time to minimize potential displacement of units during the consolidation and reconsolidation process. The 4 hours is based on typical curing conditions and may be increased based on local climatic conditions at the time of construction. For example, during cold weather construction, consider increasing the 4-hour curing period. When a wall is to be grouted with self-consolidating grout, the grout lift height is not restricted by intermediate, reinforced bond beam locations because self-consolidating grout easily flows around reinforcement (NCMA (2006); NCMA (2007)).

3.5 E. *Consolidation* — Except for self-consolidating grout, consolidation, and reconsolidation when the pour exceeds 12 in. (305 mm), is necessary to achieve complete filling of the grout space. Reconsolidation returns the grout to a plastic state and eliminates the voids resulting from the water loss from the grout by the masonry units. It is possible to have a height loss of 8 in. (203 mm) in 8 ft (2.44 m).

Consolidation and reconsolidation are normally achieved with a mechanical vibrator. A low velocity vibrator with a ¾ in. (19.1 mm) head is used. The vibrator is activated for one to two seconds in each grouted cell of hollow unit masonry. When double open-end units are used, one cell is considered to be formed by the two open ends placed together. When grouting between wythes, the vibrator is placed in the grout at points spaced 12 to 16 in. (305 to 406 mm) apart. Excess vibration does not improve consolidation and may blow out the face shells of hollow units or separate the wythes when grouting between wythes.

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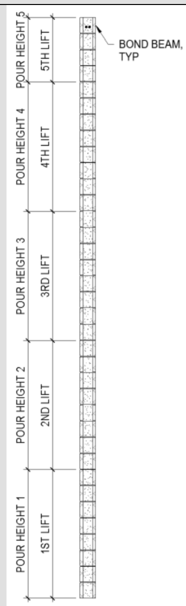
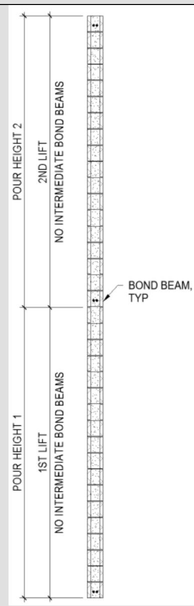
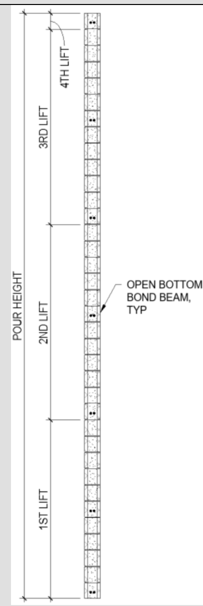
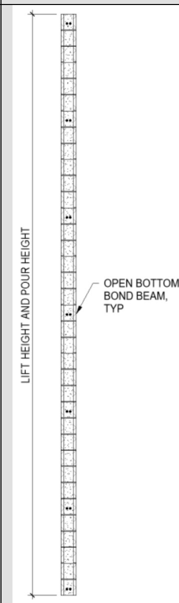
Commented [PJS104]: 20-CR-008

COMMENTARY

Type of Grouting*	Grouting with no cure time limit	Conventional grout with no intermediate bond beams	Conventional grout with intermediate bond beams	Self-consolidating grout with or without intermediate bond beams
TMS 602 Article	3.5 D.1.c 3.5 D.2.b	3.5 D.1.a	3.5 D.1.b	3.5 D.2.a
Lift Limit	5 ft-4 in.	12 ft-8 in.	See Limitation	Pour Height
Pour Height	Per Table 7	Per Table 7	Per Table 7	Per Table 7
Configuration				
Limitations	• Grout slump between 8 and 11 inches • Conventional grout or self-consolidating grout • Lift height is 1-1/2 inches less than pour height for shear key, except at top of wall.	• Masonry cured for at least 4 hours • Grout slump between 10 and 11 inches	• Masonry cured for at least 4 hours • Grout slump between 10 and 11 inches • Lift cannot exceed maximum 12 ft-8 in. • Limit grout lift to the bottom of lowest bond beam that is more than 5 ft-4 in. above bottom of grout lift • Lift height is 1-1/2 inches below the top of block for shear key, except at top of wall	• Masonry cured for at least 4 hours
Cleanouts Required	No	Yes	Yes	Yes

*Grout must conform to ASTM C476

Figure SC-20 — Grout pour height and grout lift height

COMMENTARY

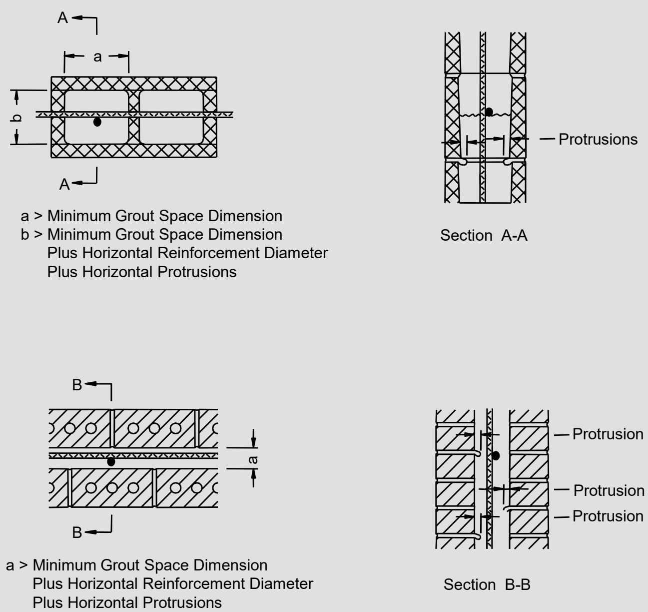


Figure SC-21 — Grout space requirements

TMS 602 SPECIFICATION

3.5 F. *Grout key* — When grouting, form grout keys between grout pours. Form grout keys between grout lifts when the first lift is permitted to set prior to placement of the subsequent lift

1. Form a grout key by terminating the grout a minimum of 1½ in. (38.1 mm) below a mortar joint.
2. Do not form grout keys within beams.
3. At beams or lintels laid with closed bottom units, terminate the grout pour at the bottom of the beam or lintel without forming a grout key.

3.5 G. *Alternate grout placement* — Place masonry units and grout using construction procedures employed in the accepted grout demonstration panel.

3.5 H. *Grouting AAC masonry* — Wet AAC masonry thoroughly before grouting to ensure that the grout flows to completely fill the space to be grouted.

COMMENTARY

3.5 F. *Grout key* — The top of a grout pour should not be located at the top of a unit, but at a minimum of 1½ in. (38 mm) below the bed joint.

If a lift of grout is permitted to set prior to placing the subsequent lift, a grout key is required within the grout pour. This setting normally occurs if the grouting is stopped for more than one hour.

TMS 602 SPECIFICATION

3.6 — Prestressing tendon installation and stressing procedure

3.6 A. *Site tolerances*

1. Tolerance for prestressing tendon placement in the out-of-plane direction in walls shall be ± $^{1}/_{4}$ in. (6.4 mm) for masonry cross-sectional dimensions less than nominal 8 in. (203 mm) and ± $^{3}/_{8}$ in. (9.5 mm) for masonry cross-sectional dimensions equal to or greater than nominal 8 in. (203 mm).
2. Tolerance for prestressing tendon placement in the in-plane direction of walls shall be ± 1 in. (25.4 mm).
3. If prestressing tendons are moved more than one tendon diameter or a distance exceeding the tolerances stated in Articles 3.6 A.1 and 3.6 A.2 to avoid interference with other tendons, reinforcement, conduits, or embedded items, notify the Architect/Engineer for acceptance of the resulting arrangement of prestressing tendons.

3.6 B. *Application and measurement of prestressing force*

1. Determine the prestressing force by both of the following methods:
   a. Measure the prestressing tendon elongation and compare it with the required elongation based on average load-elongation curves for the prestressing tendons.
   b. Observe the jacking force on a calibrated gage or load cell or by use of a calibrated dynamometer. For prestressing tendons using bars of less than 150 ksi (1034 MPa) tensile strength, Direct Tension Indicator (DTI) washers complying with ASTM F959 or ASTM F959M are acceptable.
2. Ascertain the cause of the difference in force determined by the two methods described in Article 3.6 B.1 when the difference exceeds 5 percent for pretensioned members or 7 percent for post-tensioned members, and correct the cause of the difference.
3. When the total loss of prestress due to unreplaced broken prestressing tendons exceeds 2 percent of total prestress, notify the Architect/Engineer.

COMMENTARY

3.6 — Prestressing tendon installation and stressing procedure

Installation of tendons with the specified tolerances is common practice. The methods of application and measurement of prestressing force are common techniques for prestressed concrete and masonry members. Designer, contractor, and inspector should be experienced with prestressing and should consult the Post-Tensioning Institute's Field Procedures Manual for Unbonded Single Strand Tendons (PTI (~~1994~~2016)) or similar literature before conducting the Work. Critical aspects of the prestressing operation that require inspection include handling and storage of the prestressing tendons and anchorages, installation of the anchorage hardware into the foundation and capping members, integrity and continuity of the corrosion-protection system for the prestressing tendons and anchorages, and the prestressing tendon stressing and grouting procedures.

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The design method in TMS 402 Chapter 10 is based on an accurate assessment of the level of prestress. Tendon elongation and tendon force measurements with a calibrated gauge or load cell or by use of a calibrated dynamometer have proven to provide the required accuracy. For tendons using steels of less than 150 ksi (1034 MPa) strength, Direct Tension Indicator (DTI) washers also provide adequate accuracy. These washers have dimples that are intended to compress once a predetermined force is applied on them by the prestressing force. These washers were first developed by the steel industry for use with high-strength bolts and have been modified for use with prestressed masonry. The designer should verify the actual accuracy of DTI washers and document it in the design.

Burning and welding operations in the vicinity of prestressing tendons must be carefully performed because the heat may lower the tendon strength and cause failure of the stressed tendon.

TMS 602 SPECIFICATION

3.6 C. *Grouting bonded tendons*

1. Mix prestressing grout in equipment capable of continuous mechanical mixing and agitation so as to produce uniform distribution of materials, pass through screens, and pump in a manner that will completely fill tendon ducts.
2. Maintain temperature of masonry above 35°F (1.7°C) at time of grouting and until field-cured 2 in. (50.8 mm) cubes of prestressing grout reach a minimum compressive strength of 800 psi (5.52 MPa).
3. Keep prestressing grout temperatures below 90°F (32.2°C) during mixing and pumping.

3.6 D. *Burning and welding operations* — Carefully perform burning and welding operations in the vicinity of prestressing tendons so that tendons and sheathings, if used, are not subjected to excessive temperatures, welding sparks, or grounding currents.

3.6 E. *Prestressing anchorages, couplers, and end blocks*

1. Place couplers where accepted by Architect/Engineer. Enclose with housing that permits anticipated movements of the couplers during stressing.
2. Protect anchorages, couplers, and end fittings against corrosion.
3. Protect exposed anchorages, couplers, and end fittings to achieve the fire-resistance rating required for the element by the legally adopted building code.

3.7 — Field quality control

3.7 A. Verify f'_m and f'_{AAC} in accordance with Article 1.6.

3.7 B. Sample and test grout as required by Articles 1.4 B and 1.6.

3.8 — Cleaning

Clean exposed masonry surfaces of stains, efflorescence, mortar and grout droppings, and debris using methods that do not damage the masonry.

COMMENTARY

3.6 E. *Prestressing anchorages, couplers, and end blocks* — Protection of anchorage devices typically includes filling the opening of bearing pads with grease, grouting the recess in bearing pads, and providing drainage of cavities housing prestressing tendons with base flashing and weep holes.

When anchorages and end fittings are exposed, additional precautions to achieve the required fire ratings and mechanical protection for these elements must be taken.

3.7 — Field quality control

3.7 A. The specified frequency of testing must equal or exceed the minimum requirements of the quality assurance tables.

3.7 B. ASTM C1019 requires a mold for the grout specimens made from the masonry units that will be in contact with the grout. Thus, the water absorption from the grout by the masonry units is simulated. Sampling and testing frequency may be based on the volume of grout to be placed rather than the wall area. Alternative forming methods can also be used provided a conversion factor based on comparative testing of 10 sets of specimens has been established as required by ASTM C1019, Section 6.2

3.8 — Cleaning

Use of undiluted cleaning products, especially acids, and failing to pre-wet the masonry or to adequately rinse the masonry after cleaning can cause damage. In some situations, cleaning without chemicals may be appropriate.

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FOREWORD TO SPECIFICATION CHECKLISTS

TMS 602 SPECIFICATION

F1. This Foreword is included for explanatory purposes only; it does not form a part of TMS 602.

F2. TMS 602 may be referenced by the Architect/Engineer in the Project Specification for any building project, together with supplementary requirements for the specific project. Responsibilities for project participants must be defined in the Project Specification.

F3. Checklists do not form a part of TMS 602. Checklists are provided to assist the Architect/Engineer in selecting and specifying project requirements in the Project Specification. The checklists identify the Articles and paragraphs of TMS 602 and the action required or available to the Architect/Engineer.

F4. The Architect/Engineer must make adjustments to the Specification based on the needs of a particular project by reviewing each of the items in the checklists and including the items the Architect/Engineer selects as mandatory requirements in the Project Specification.

F5. The Mandatory Requirements Checklist indicates work requirements regarding specific qualities, procedures, materials, and performance criteria that are not defined in TMS 602 or requirements for which the Architect/Engineer must define which of the choices apply to the project.

F6. The Optional Requirements Checklist identifies Architect/Engineer choices and alternatives.

COMMENTARY

F1. No Commentary

F2. Building codes (of which TMS 602 is a part by reference) set minimum requirements necessary to protect the public. Project specifications may stipulate requirements more restrictive than the minimum. Adjustments to the needs of a particular project are intended to be made by the Architect/Engineer by reviewing each of the items in the Checklists and then including the Architect/Engineer's decision on each item as a mandatory requirement in the project specifications.

F3. The Checklists are addressed to each item of this Specification where the Architect/Engineer must or may make a choice of alternatives; may add provisions if not indicated; or may take exceptions. The Checklists consist of two columns; the first identifies the articles and paragraphs of TMS 602, and the second column contains notes to the Architect/Engineer to indicate the type of action that may be required by the Architect/Engineer. Checklist items that are not applicable to a project should not be included in the Project Specifications.

MANDATORY REQUIREMENTS CHECKLIST

TMS 602 Article/Paragraph		Notes to the Architect/Engineer
PART 1 — GENERAL		
1.4 A	Compressive strength requirements	Specify f'_m and f'_{AAC}, except for veneer, glass unit masonry, and prescriptively designed partition walls. Specify f'_{mi} for prestressed masonry.
1.4 B.2	Unit strength method	Specify when strength of grout is to be determined by test.
1.5	Submittals	Define the submittal reporting and review procedure.
1.6 A.1	Testing Agency's services and duties	Specify which level in Table 3 applies to each portion of the project. Specify which portions of the masonry were designed in accordance with the prescriptive partition wall, veneer, or glass unit masonry provisions of TMS 402 and are, therefore, exempt from verification of f'_m.
1.6 B.1	Inspection Agency's services and duties	Specify which level in Tables 3 and 4 applies to each portion of the project. Specify which portions of the masonry were designed in accordance with the prescriptive partition wall, veneer, or glass unit masonry provisions of TMS 402 and are, therefore, exempt from verification of f'_m.
1.6 D	Sample panels	Specify requirements for sample panels.
PART 2 — PRODUCTS		
2.1	Mortar materials	Specify type, color, and cementitious materials to be used in mortar and mortar to be used for the various parts of the project and the type of mortar to be used with each type of masonry unit.
2.3	Masonry unit materials	Specify the masonry units to be used for the various parts of the projects.
2.4	Reinforcement, prestressing tendons, and metal accessories	Specify type and grade of reinforcement, tendons, connectors, and accessories.
2.4 A	Reinforcing Steel	When deformed reinforcing bars conforming to ASTM A615/A615M or ASTM A996/A996M are required by strength design in accordance with TMS 402 Chapter 9 or Chapter 11, specify that the actual yield strength must not exceed the specified yield strength multiplied by 1.3.
2.4 D	Joint reinforcement	Specify joint reinforcement wire size and number of longitudinal wires when joint reinforcement is to be used as shear reinforcement.
2.4 ~~I~~K	Stainless steel	Specify when stainless steel joint reinforcement, anchors, ties, and/or accessories are required.
2.4 ~~J~~L	Coating for corrosion protection	Specify the types of corrosion protection that are required for each portion of the masonry construction.
2.4 ~~K~~M	Corrosion protection for tendons	Specify the corrosion protection method.

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MANDATORY REQUIREMENTS CHECKLIST (Continued)

TMS 602 Article/Paragraph	Notes to the Architect/Engineer
2.4 ~~L~~N Prestressing anchorages, and couplers	Specify the anchorages and couplers and their corrosion protection.
2.5 E Joint fillers	Specify size and shape of joint fillers.
2.7C Prefabricated masonry	Specify prefabricated masonry and requirements in supplement of those of ASTM C901.
PART 3 — EXECUTION	
3.3 ~~D~~E.2-4 Pipes and conduits	Specify sleeve sizes and spacing.
3.3 ~~D~~E.5 Accessories	Specify accessories not indicated on the project drawings.
3.3 ~~D~~E.6 Movement joints	Indicate type and location of movement joints on the project drawings and, where necessary, indicate where movement joints are not permitted in order to maintain structural design intent.
3.4 Reinforcement, tie and anchor installation	Specify type and location of reinforcement, ties and anchors on the project drawings
3.4 B.12 Placement tolerances	Indicate *d* distance for beams on drawings or as a schedule in the project specifications.
3.4 ~~E~~ D Veneer Ties	Specify the type of tie required.

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OPTIONAL REQUIREMENTS CHECKLIST

TMS 602 Article/Paragraph		Notes to the Architect/Engineer
PART 1 — GENERAL		
1.4 A	Compressive strength requirements	Specify f'_m when required by an engineered design of anchored masonry veneer to meet the requirements of Section 13.2.3.
1.5 B	Submittals	Specify additional required submittals. Specify when submittal of qualifications of masonry testing laboratory is required
1.6	Quality assurance	Define who will retain the Testing Agency and Inspection Agency, if other than the Owner.
1.6A	Testing Agency's services and duties	Identify items beyond the scope of Tables 3 and 4 that require testing and evaluation for the project.
1.6 B	Inspection Agency's services and duties	Identify items beyond the scope of Tables 3 and 4 that require inspection and evaluation for the project.
PART 2 — PRODUCTS		
2.2	Grout Materials	Specify grout requirements at variance with TMS 602. Specify admixtures.
2.5 A and 2.5 B	Movement joint	Specify requirements at variance with TMS 602.
2.5 D	Masonry cleaner	Specify where acid or caustic solutions are allowed and how to neutralize them.
2.6 A	Mortar	Specify if hand mixing is allowed and the method of measurement of material. Specify when mortar cementitious materials are limited to mortar cement or non-air-entrained cement-lime, and the masonry members to which the limitation applies.
2.6 B	Grout	Specify requirements at variance with TMS 602.
PART 3 — EXECUTION		
3.2 B	Foundation preparation	Specify when an unfinished or roughened base surface is required.
3.2 C	Wetting masonry units	Specify when units are to be wetted.
3.3 A	Bond pattern	Specify bond pattern if not running bond.
3.3 B.2	Bed and head joints	Specify thickness and tooling differing from TMS 602.
3.3 B.3	Hollow units	Specify when cross webs are to be mortar bedded.
3.3 B.4	Solid units	Specify mortar bedding at variance with TMS 602.
3.3 B.6	Glass units	Specify mortar bedding at variance with TMS 602.
3.3 B.8	AAC Masonry	Specify when mortar may be omitted from AAC running bond masonry head joints that are less than 8 in. (200 mm) (nominal) tall.
3.3 C	Adhered veneer units	Specify installation procedures when units having a bonded area greater than 360 in.^{2} (0.232 m^2) are used.

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OPTIONAL REQUIREMENTS CHECKLIST

TMS 602 Article/Paragraph		Notes to the Architect/Engineer
3.3 C.4.a	Weep screed	Specify if something other than a weep screed is used.
3.3 C.1	Collar joints	Specify the filling of collar joints less than $^{3}/_{4}$ in. (19.1 mm) thick differing from TMS 602.
3.3 D.4.d	Tooling	Specify when mortar is not required between units.
3.3 E	Embedded items and accessories	Specify locations where sleeves are required for pipes or conduits.
3.4 C	Wall ties	Specify requirements at variance with TMS 602.
3.4 ~~E.~~D	Veneer ties	Specify requirements at variance with TMS 602.

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REFERENCES FOR THE SPECIFICATION COMMENTARY

References, Part 1

Atkinson, R.H., and Kingsley, G.R. (1985). "A Comparison of the Behavior of Clay and Concrete Masonry in Compression," Atkinson-Noland & Associates, Inc., Sept. 1985.

Bennett, R.M. (2010). "Proposed Masonry Specified Compressive Strength Requirements." *Journal of ASTM International*, ASTM International, 7(2), Paper JAI102663.

BIA TN 1 (~~1992~~2018). "~~All~~ Hot and Cold Weather Construction," *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

BIA TN 39 (2001). "Testing for Engineered Brick Masonry—Brick and Mortar," *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

BIA TN 39B (1988). "Testing for Engineered Brick Masonry—Quality Control," *Technical Notes on Brick Construciton*, Brick Industry Association, www.gobrick.com.

Brown, R.H., and Borchelt, J.G. (1990). "Compression Tests of Hollow Brick Units and Prisms," *Masonry Components to Assemblages*, STP 1063, ASTM International, 263 - 278.

Chrysler, J. (~~2010~~2017). *Reinforced Concrete Masonry Construction Inspector's Handbook*, ~~7th~~ 10th Edition, Masonry Institute of America and International Code Council.

Drysdale, R.G., Hamid, A.A., and Baker, L.R. (1999). *Masonry Structures: Behavior and Design*, 2nd edition, The Masonry Society.

Farny, J. A., Melander, J. M., and Panarese, W.C. (2008). *Concrete Masonry Handbook for Architects, Engineers, and Builders*, Portland Cement Association.

Hegemier, G.A., Krishnamoorthy, G., Nunn, R.O., and Moorthy, T.V. (1978). "Prism Tests for the Compressive Strength of Concrete Masonry," *North American Masonry Conference*, Boulder, CO, The Masonry Society, 18-1 through 18-17.

IMI (1973). "Recommended Practices and Guide Specifications for Cold Weather Masonry Construction," International Masonry Industry All-Weather Council, Washington, DC.

Masonry Industry Council (1999) "~~hot~~ Hot& Cold Weather Masonry Construction", Masonry Contractors Association of America, ~~Lonbard~~Lombard, IL.

Miller, D.E. Noland, J.L. and Feng, C.C. (1979). "Factors Influencing the Compressive Strength of Hollow Clay Unit Prisms," *5th International Brick Masonry Conference*, Washington DC.

NCMA (1994). "Research Evaluation of Flexural Tensile Strength of Concrete Masonry," *Resarch Report MR 10*, National Concrete Masonry Association.

NCMA (2008). "Inspection and Testing of Concrete Masonry Construction," National Concrete Masonry Association and International Code Council.

NCMA (2012). "Recalibration of the Unit Strength Method for Verifying Compliance with the Specified Compressive Strength of Concrete Masonry," *Research Report MR37*, National Concrete Masonry Association.

Noland, J.L. (1982). "Proposed Test Method for Determining Compressive Strength of Clay-Unit Prisms," Atkinson-Noland & Associates, Inc., Boulder, CO, June 1982.

~~Panarese, W.C., S.H. Kosmatka, S.H., and F.A. Randal, F.A., Jr. (1991). *Concrete Masonry Handbook for Architects, Engineers, and Builders*, Portland Cement Association, 121-123.~~

PCA (~~1993~~2006). "Hot Weather Masonry Construction," *Trowel Tips*, IS243, Portland Cement Association.

Priestley, M.J.N., and Elder, D.M. (1983). "Stress-Strain Curves for Unconfined and Confined Concrete Masonry," *Journal of the American Concrete Institute*, American Concrete Institute, 80(3), 192-201.

SCI and MIA (2006). "CodeMaster, Special Inspection for Masonry," Structures & Codes Institute and Masonry Institute of America.

SCPI (1969). *Recommended Practice for Engineered Brick Masonry*, Brick Industry Association (formerly Structural Clay Products Institute).

Tomasetti, A.A. (1990). "Problems and Cures in Masonry," *Masonry Components to Assemblages*, STP 1063, ASTM International, 324-338.

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References, Part 2

ACI 315R (~~1999~~2018). ~~*Details and Detailing of Concrete Reinforcement*~~*Guide to Presenting Reinforcing Steel Design Details*, American Concrete Institute.

ACI-SEASC (1982). "Test Report on Slender Walls," ACI Southern California Chapter/Structural Engineers Association of Southern California.

ATL (1981). Unpublished Field Test Report, File 80-617, B'Nai B'Rith Housing, Associated Testing Laboratories, Houston, TX, 1981.

Beall, C. (1989). "Tips on Designing, Detailing, and Specifying Glass Block Panels," *Masonry Construction*, The Aberdeen Group, March 1989, 92 - 99.

BIA TN 18A (~~2006~~2019). "Accommodating Expansion of Brickwork", *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

BIA TN 18 (~~2006~~2019). "Volume Changes – Analysis and Effects of Movement," *Technical Notes on Brick Construction*, Brick Industry Association, www.gobrick.com.

Catani, M.J. (1985). "Protection of Embedded Steel in Masonry," *Construction Specifier*, Construction Specifications Institute, 38(1), 62.

Garrity, S.W. (1995). "Corrosion Protection of Prestressing Tendons for Masonry," *Seventh Canadian Masonry Symposium*, Hamilton, Ontario, 736-750.

Glashaus (1992). *WECK Glass Blocks*, Glashaus Inc.

Grimm, C.T. (1985). "Corrosion of Steel in Brick Masonry," *Masonry: Research, Application, and Problems*, STP-871, ASTM International, 67-87.

Grimm, C.T. (1988). "Masonry Cracks: A Review of the Literature," *Masonry: Materials, Design, Construction, and Maintenance*, STP-992, ASTM International, 257-280.

Li, D., and Neis, V.V. (1986). "The Performance of Reinforced Masonry Beams Subjected to Reversal Cyclic Loadings," *4th Canadian Masonry Symposium*, Fredericton, New Brunswick, 1, 351-365.

NCMA TEK 10-~~2C~~2D (~~2010~~2019). "Control Joints for Concrete Masonry Walls-Empirical Method," *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

NCMA TEK 12-04D (2006). "Steel Reinforcement for Concrete Masonry," *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

Pittsburgh Corning (1992). "PC Glass Block Products," *GB 185*, Pittsburgh Corning Corp.

PTI (2006). "Chapter 4-Specifying Post-Tensioning," *Post-Tensioning Manual*, 6th Edition, Post-Tensioning Institute, 73-79.

Schultz, A.E. and Scolforo, M.J. (1991). "An Overview of Prestressed Masonry," *TMS Journal*, The Masonry Society, 10(1), 6-21.

Soric, Z., and Tulin, L. (1987), "Bond and Splices in Reinforced Masonry," *TCCMAR Report No. 6.2-2*, The Masonry Society.

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Commented [PJS9]: 20-EX-002

Commented [PJS10]: 20-EX-002

References, Part 3

ACI 117 (1990). *Standard Specifications for Tolerances for Concrete Construction and Materials*, American Concrete Institute.

ACI 440.1R (2015). *Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced Polymer ~~(FRP)~~ Bars*, American Concrete Institute.

ASTM C1780 (~~2017~~2020). "Standard Practice for Installation Methods for Cement-based Adhered ~~Manufactured Stone~~ Masonry Veneer", ASTM International, www.astm.org.

BIA TN28C (2014). "Thin Brick Veneer," *Technical Notes on Brick Construction*, ~~"28C Thin Brick Veneer,"~~ Brick Industry Association."

Chrysler, J. (~~2010~~2017). *Reinforced Concrete Masonry Construction Inspector's Handbook*, ~~7th~~ 10th Edition, Masonry Institute of America and International Code Council.

CRSI (2015), "Frequently Asked Questions (FAQ) About Reinforcing Bars," *Construction Technical Note CTN-G-2-15*, Concrete Reinforcing Steel Institute. resources.crsi.org/index.cfm/_api/render/file/?method=inline&fileID=2A6B7DB0-D41F-D5F2-835D7F25685989A3

MCAA (2012). *Standard Practice for Bracing Masonry Walls Under Construction*, Mason Contractors Association of America.

~~MVMA~~ NCMA (~~2017~~2021). *Installation Guide and Detailing Options for Compliance with ASTM C1780*, 5th edition, 5th printing, ~~Masonry Veneer Manufacturers Association~~National Concrete Msaonry Association.

NCMA (2006). "Self-Consolidating Grout Investigation: Compressive Strength, Shear Bond, Consolidation and Flow," *Research Report MR 29*, National Concrete Masonry Association.

NCMA (2007). "Self-Consolidating Grout Investigation: Making and Testing Prototype SCG Mix Designs – Report of Phase II Research," *Research Report MR31*, National Concrete Masonry Association.

NCMA TEK 03-2A (2005). "Grouting Concrete Masonry Walls", *e-TEK Notes*, National Concrete Masonry Association, www.ncma.org.

OSHA (1994). "Safety and Health Regulations for Construction, Concrete and Masonry Construction," Occupational Safety & Health Administration, https://www.osha.gov/pls/oshaweb/owadisp.show_document?p_table=STANDARDS&p_id=10785.

PTI (~~1994~~2016). *Field Procedures Manual for Unbonded Single Strand Tendons*, ~~2nd~~3rd Edition, Post-Tensioning Institute.

Stecich, J.P., Hanson, J.M. and Rice, P.F. (1984). "Bending and Straightening of Grade 60 Reinforcing Bars" *Concrete International*, American Concrete Institute, 6(8), 14-23.

UBC (1985). *Uniform Building Code*, International Conference of Building Officials.

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