Building Code Requirements for Masonry Structures
(ACI 530-~~05~~-xx/ASCE 5-~~05~~-xx/TMS 402-~~05~~-xx)
Reported by the Masonry Standards Joint Committee (MSJC)

CHAPTER 1

GENERAL DESIGN REQUIREMENTS FOR MASONRY

1.1 — Scope

1.1.1 *Minimum requirements*

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

1.1.2 *Governing building code*

This Code supplements the legally adopted building code and shall govern in ~~all~~ matters pertaining to structural design and construction of masonry elements, except where this Code is in conflict with requirements in the legally adopted building code. In areas without a legally adopted building code, this Code defines the minimum acceptable standards of design and construction practice.

1.1.3 *Design procedures*

Masonry structures and their component members shall be designed in accordance with the provisions of this Chapter and one of the following:

(a) Allowable Stress Design of Masonry: Chapter 2.

(b) Strength Design of Masonry: Chapter 3.

(c) Prestressed Masonry: Chapter 4.

(d) Empirical Design of Masonry: Chapter 5.

(e) Veneer: Chapter 6.

(f) Glass Unit Masonry: Chapter 7.

(g) Strength Design of Autoclaved Aerated Concrete (AAC) Masonry: Appendix A.

1.1.4 *SI equivalents*

SI values shown in parentheses are not part of this Code. The equations in this document are for use with the specified inch-pound units only. The equivalent equations for use with SI units are provided in Conversion of Units on Page C-66.

1.2 — Contract documents and calculations

1.2.1 Project drawings and project specifications for masonry structures shall identify the individual responsible for their preparation.

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

(a) Name and date of issue of code and supplement to which the design conforms.

(b) ~~All l~~Loads used in the design of masonry.

(c) Specified compressive strength of masonry at stated ages or stages of construction for which masonry is designed, except where specifically exempted by Code provisions.

(d) Size and location of structural elements.

(e) Details of anchorage of masonry to structural members, frames, and other construction, including the type, size, and location of connectors.

(f) Details of reinforcement, including the size, grade, type, and location of reinforcement.

(g) Reinforcing bars to be welded and welding requirements.

(h) Provision for dimensional changes resulting from elastic deformation, creep, shrinkage, temperature, and moisture.

(i) Size and location of conduits, pipes, and sleeves.

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 ~~1.15~~1.17~~1~~1.18, or shall include an itemized quality assurance program that equals or exceeds the requirements of Section ~~1.15~~1.17~~1~~1.18.

1.3 — Approval of special systems of design or construction

Sponsors of any system of design or construction within the scope of this Code, the adequacy of which has been shown by successful use or by analysis or test, but that does not conform to or is not covered by this Code, shall have the right to present the data on which their design is based to a board of examiners appointed by the building official. The board shall be composed of registered

engineers and shall have authority to investigate the data so submitted, require tests, and formulate rules governing design and construction of such systems to meet the intent of this Code. The rules, when approved and promulgated by the building official, shall be of the same force and effect as the provisions of this Code.

1.4 — Standards cited in this Code

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

ACI 530.1-~~05~~-xx/ ASCE 6-~~05~~-xx/ TMS 602-~~05~~-xx — Specification for Masonry Structures

~~ASCE 7-93 — Minimum Design Loads for Buildings and Other Structures~~

ASCE 7-~~02~~ 05 — Minimum Design Loads for Buildings and Other Structures

ASTM A 416/A 416M-05 ~~02~~ — Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete

ASTM A 421/A 421M-05 ~~02~~ — Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete

ASTM A 722/A 722M-00 (2005) ~~98~~ — Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete

ASTM C 34-05 — Specification for Structural Clay Load-bearing Wall Tile

ASTM C 426-05 ~~99~~ — Test Method for Linear Drying Shrinkage of Concrete Masonry Units

ASTM C 476-02 — Specification for Grout for Masonry

ASTM C 482-02 — Test Method for Bond Strength of Ceramic Tile to Portland Cement

ASTM C 1006-84(2001) Test Method for Splitting Tensile Strength of Masonry Units

ASTM C 1386-98 Specification for Precast Autoclaved Aerated Concrete (PAAC) Wall Construction Units

ASTM C 1611/C 1611M-05 Standard Test Method for Slump Flow of Self-Consolidating Concrete

ASTM E 111-04 97 — Test Method for Young's Modulus, Tangent Modulus, and Chord Modulus

ASTM E 488-96 (2003) — Test Methods for Strength of Anchors in Concrete and Masonry Elements

AWS D 1.4-05 98 Structural Welding Code — Reinforcing Steel

1.5 — Notation

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

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

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

A_n = net cross-sectional area of a member~~masonry~~, 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 ~~for calculating tensile breakout capacity of anchor bolts~~, in.^{2} (mm^2)

A_{pv} = projected shear area on masonry surface of one-half of a right circular cone ~~for calculating shear breakout capacity of anchor bolts~~, in.^{2} (mm^2)

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

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

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

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

A_{st} = total area of laterally tied longitudinal reinforcing steel ~~in a reinforced masonry column or pilaster~~, in.^{2} (mm^2)

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

B_a = allowable axial ~~force~~ 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)

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

B_v = allowable shear ~~force~~ 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 steel yielding, 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 steel yielding, lb (N)

b = width of section, in. (mm)

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

b_{af} = factored axial force in an anchor bolt, lb (N)

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

b_{vf} = factored shear force in an anchor bolt, lb (N)

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

C_d = deflection amplification factor

~~C_n = nominal bearing strength, lb (N)~~

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

D = dead load or related internal moments and forces

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

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

d_v = actual depth of a member~~masonry~~ 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_m = modulus of elasticity of masonry in compression, psi (MPa)

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

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

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 = lateral pressure of liquids or related internal moments and forces

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

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

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

F_v = allowable shear stress in masonry, 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'_{AAC} = specified compressive strength of AAC masonry~~, the minimum compressive strength for a class of AAC as specified in ASTM C 1386~~, psi (MPa)

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

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

f'_{mi} = specified compressive strength of 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 all prestress losses have occurred, psi (MPa)

$f_{t\,AAC}$ = splitting tensile strength of AAC as determined in accordance with ASTM C 1006, psi (MPa)

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

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

H = lateral pressure of soil or related internal moments and forces

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

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

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 ~~the least of the masonry cover, the clear spacing between adjacent~~ reinforcement development, ~~, or and 5 times~~multiplied by ~~d_b,~~ in. (mm)

K_{AAC} = Dimension used to calculate ~~the least of the grout cover, the clear spacing between adjacent~~ reinforcement development for AAC masonry, ~~or and 5 times~~multiplied by ~~d_b,~~ in. (mm)

k_c = coefficient of creep of masonry, per psi (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)

L = live load or related internal moments and forces

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

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

l_{be} = anchor bolt edge distance~~, measured in the direction of load, from edge of masonry to center of the cross section of anchor bolt~~, in. (mm)

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

l_e = equivalent embedment length provided by standard hooks measured from the start of the hook (point of tangency), 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 loading for which deflection is computed, in.-lb (N-mm)

M_c = factored moment magnified for the effects of member curvature, in.-lb (N-mm)

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

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

M_{ser} = service moment at midheight of a member, including P-delta effects, in.-lb (N-mm)

M_u = factored moment, in.-lb (N-mm)

n = modular ratio, E_s/E_m

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

N_v = compressive force acting normal to shear surface, lb (N)

P = axial load, lb (N)

P_a = allowable axial compressive force in a reinforced ~~masonry due to axial load~~member, lb (N)

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 = factored axial load, lb (N)

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

P_{uw} = factored weight of wall area tributary to wall section under consideration, lb (N)

Q = first moment about the neutral axis of ~~a section of that portion of the cross section lying between the neutral axis and extreme fiber~~ 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 (earthquake-induced) forces

R = ~~seismic~~ response modification ~~factor~~coefficient

r = radius of gyration, in. (mm)

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 C 426

T = forces and moments caused by restraint of temperature, shrinkage, and creep strains or differential movements

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

v = shear stress, psi (MPa)

V = shear force, lb (N)

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

~~V_m = shear strength provided by masonry, lb (N)~~

V_n = nominal shear strength, 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 = factored shear force, lb (N)

W = wind load or related internal moments and forces

W_S = dimension of the structural wall strip defined in Section 5.5.1 and shown in Figure 5.5.1-1.

W_T = dimension of the tributary length of wall, defined in Section 5.5.1 and shown in Figure 5.5.1-1.

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

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

β = 0.25 for fully grouted masonry or 0.15 for other than fully grouted masonry

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

γ = reinforcement size factor

Δ = calculated story drift, in. (mm)

Δ_a = allowable story drift, in. (mm)

δ = moment magnification factor

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

δ_s = horizontal deflection at midheight under service loads, in. (mm)

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

ε_{cs} = drying shrinkage of AAC~~, defined as the difference in the relative change in length between the moisture contents of 30% and 6%~~

ε_{mu} = maximum usable compressive strain of masonry

μ_{AAC} = coefficient of friction of AAC

ϕ = strength-reduction factor

ρ = reinforcement ratio

ρ_{max} = maximum flexural tension reinforcement ratio

1.6 — Definitions

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

AAC masonry — Masonry made of autoclaved aerated concrete (AAC) units, manufactured without internal reinforcement, and bonded together using thin- or thick-bed mortar.

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.

Architect/Engineer — The architect, engineer, architectural firm, engineering firm, or architectural and engineering firm issuing drawings and specifications, or administering the work under contract specifications and project drawings, or both.

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

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

Autoclaved aerated concrete — Low-density cementitious product of calcium silicate hydrates, whose material specifications are defined in ASTM C 1386.

Backing — The wall or surface to which the veneer is ~~secured~~attached.

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

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

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

~~*Camber* — A deflection that is intentionally built into a structural element to improve appearance or to nullify compensate for the deflection of the element under the effects of loads, shrinkage, and creep.~~

Cavity wall — A multiwythe noncomposite masonry wall with a continuous air space within the wall (with or without insulation), which is tied together with metal ties.

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

Column — An isolated vertical member whose horizontal dimension measured at right angles to its thickness does not exceed 3 times its thickness and whose height is greater than 4 times its thickness.

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 — Multicomponent masonry members acting with 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 ACI 530.1/ASCE 6/TMS 602.

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

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

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

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, calculated by multiplying the deflections determined from an elastic analysis by the appropriate deflection amplification factor, C_d, from ASCE 7-~~02~~.

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

Diaphragm — A roof or floor system designed to transmit lateral forces to shear walls or other lateral load 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. Thickness is given first, followed by height and then length.

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

Effective height — Clear height of a braced member between lateral supports 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.

Foundation pier — An isolated vertical foundation member whose horizontal dimension measured at right angles to its thickness does not exceed 3 times its thickness and whose height is equal to or less than 4 times its thickness.

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

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

Grout, self-consolidating — A highly fluid and stable grout typically with admixtures, that ~~can flow around reinforcement and consolidate within masonry under its own weight without consolidation, while retaining its homogeneity~~ remains homogeneous when placed and does not require puddling or vibration for consolidation.

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

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

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.

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, service — Load specified by the legally adopted building code.

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

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

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

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

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

Pier — An isolated vertical member whose horizontal dimension measured at right angles to its thickness is not less than 3 times its thickness nor greater than 6 times its thickness and whose height is less than 5 times its length.

Post-tensioning — Method of prestressing in which a prestressing tendon is tensioned after the masonry has been placed.

Prestressed masonry — Masonry in which internal stresses have been introduced to counteract stresses resulting from applied loads.

Pretensioning — Method of prestressing in which a prestressing tendon is tensioned before the transfer of stress into the masonry.

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

Prestressing tendon — Steel elements such as wire, bar, or strand, used to impart prestress to masonry.

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

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

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

Reinforcement — Nonprestressed steel reinforcement.

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

Required strength — The strength needed to resist factored loads.

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

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

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

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

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

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

Shear wall, ordinary plain (unreinforced) 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.

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 ASTM C 1611/C 1611M.

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

Specified compressive strength of AAC masonry, f'_{AAC} — Minimum compressive strength, expressed as force per unit of net cross-sectional area, required of the AAC masonry used in construction by the contract documents, and upon which the project design is based. Whenever the

quantity f'_{AAC} is under the radical sign, the square root of numerical value only is intended and the result has units of 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 contract documents, and upon which the project design is based. Whenever the quantity f'_m is under the radical sign, the square root of numerical value only is intended and the result has units of psi (MPa).

Stack bond — For the purpose of this Code, stack bond is other than running bond. Usually the placement of units is such that the head joints in successive courses are vertically aligned.

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

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

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

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

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

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.

Thin-bed mortar — Mortar for use in construction of AAC unit masonry whose joints are 0.06 in. (1.5 mm) or less.

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

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 axis of the member.

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

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

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

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

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

Visual stability index (VSI) — An index, defined in ASTM C 1611/C 1611M, that qualitatively indicates the stability of self-consolidating grout

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

Wall, loadbearing — Wall carrying vertical loads greater than 200 lb/lineal ft (2919 N/m) in addition to its own weight.

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

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

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

1.7 — Loading

1.7.1 *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.

1.7.2 *Load provisions*

~~Service~~ 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 ~~service~~ design loads in the legally adopted building code, the load provisions of ASCE 7-~~93~~ shall be used, except as noted in this Code.

1.7.3 *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.

1.7.4 *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.

1.7.5 *Lateral load distribution*

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

1.7.5.1 Flanges of intersecting walls designed in accordance with Section 1.9.4.2 shall be included in stiffness determination.

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

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

1.8 — Material properties

1.8.1 *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.

1.8.2 *Elastic moduli*

1.8.2.1 *Steel reinforcement*

E_s = 29,000,000 psi (200,000 MPa)

1.8.2.2 *Clay and concrete masonry*

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

$E_m = 700\, f'_m$ for clay masonry;

$E_m = 900\, f'_m$ for concrete masonry;

or the chord modulus of elasticity taken between 0.05 and 0.33 of the maximum compressive strength of each prism determined by test in accordance with the prism test method, Article 1.4 B.3 of ACI 530.1/ASCE 6/TMS 602, and ASTM E 111.

1.8.2.2.2 $E_v = 0.4E_m$

1.8.2.3 *AAC masonry*

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

$E_{AAC} = 6500\ (f'_{AAC})^{0.6}$

1.8.2.3.2 $E_V = 0.4\ E_{AAC}$

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

1.8.3 *Coefficients of thermal expansion*

1.8.3.1 *Clay masonry*

$k_t = 4 \times 10^{-6}$ in./in./°F (7.2×10^{-6} mm/mm/°C)

1.8.3.2 *Concrete masonry*

$k_t = 4.5 \times 10^{-6}$ in./in./ °F

(8.1×10^{-6} mm/mm/°C)

1.8.3.3 *AAC masonry*

$k_t = 4.5 \times 10^{-6}$ in./in./ °F

(8.1×10^{-6} mm/mm/°C)

1.8.4 *Coefficient of moisture expansion for clay masonry*

$k_e = 3 \times 10^{-4}$ in./in. (3×10^{-4} mm/mm)

1.8.5 *Coefficient of shrinkage ~~for concrete masonry~~*

1.8.5.1 *Concrete masonry*

$k_m = 0.5\ s_l$

1.8.5.2 *AAC masonry*

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

where s_{cs} is determined in accordance with ASTM C 1386.

1.8.6 *Coefficients of creep*

1.8.6.1 *Clay masonry*

$k_c = 0.7 \times 10^{-7}$, per psi (0.1×10^{-4}, per MPa)

1.8.6.2 *Concrete masonry*

$k_c = 2.5 \times 10^{-7}$, per psi (0.36×10^{-4}, per MPa)

1.8.6.3 *AAC masonry*

$k_c = 5.0 \times 10^{-7}$, per psi (0.72×10^{-4}, per MPa)

1.8.7 *Prestressing steel*

Modulus of elasticity shall be determined by tests. For prestressing steels not specifically listed in ASTM A 416, A 421, or A 722, tensile strength and relaxation losses shall be determined by tests.

1.9 — Section properties

1.9.1 *Stress computations*

1.9.1.1 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.

1.9.1.2 In members designed for composite action, stresses shall be computed 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.

1.9.2 *Stiffness*

~~Determination~~ Computation of stiffness based on uncracked section is permissible. Use of the average net cross-sectional area of the member considered in stiffness computations is permitted.

1.9.3 *Radius of gyration*

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

1.9.4 *Intersecting walls*

1.9.4.1 Wall intersections shall meet one of the following requirements:

(a) Design shall conform to the provisions of Section 1.9.4.2.

(b) Transfer of shear between walls shall be prevented.

1.9.4.2 *Design of wall intersection*

1.9.4.2.1 Masonry shall be in running bond.

1.9.4.2.2 Flanges shall be considered effective in resisting applied loads.

1.9.4.2.3 The width of flange considered effective on each side of the web shall be the ~~lesser~~ smaller of ~~6 times the flange thickness the flange thickness or~~ the actual flange on either side of the web wall or the following: ~~-~~

(a) 6 multiplied by the nominal flange thickness, when the flange is in compression

(b) 6 multiplied by the nominal flange thickness for unreinforced masonry, when the flange is in flexural tension

(c) 0.75 multiplied by the floor to floor wall height for reinforced masonry, when the flange is in flexural tension.

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

1.9.4.2.4 Design for shear, including the transfer of shear at interfaces, shall conform to the requirements of Section 2.2.5; or Section 2.3.5; or Sections 3.1.3 and 3.3.4.1.2; or Sections 3.1.3 and 3.2.4; or Section 4.6.

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

(a) At least fifty percent of the masonry units at the interface shall interlock.

(b) Walls shall be anchored by steel connectors grouted into the wall and meeting the following requirements:

(1) Minimum size: $^{1}/_{4}$ 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 ~~in intersecting walls~~ at a maximum spacing of 48 in. (1219 mm) on centers. ~~Bond beams shall be reinforced and t~~The area of reinforcement shall not be less than 0.1 in.^{2} per ft (211 mm^2/m) ~~of wall~~~~times~~multiplied by the

vertical spacing of the bond beams in feet (meters). Reinforcement shall be developed on each side of the intersection.

1.9.5 *Bearing area*

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

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

(b) $2A_1$

The area, A_2, is the area of the lower base of the largest frustum of a right pyramid or cone that has the loaded area, A_1, as its upper base, slopes at 45 degrees from the horizontal, and is wholly contained within the support. For walls in other than running bond, area A_2 shall terminate at head joints.

1.9.6 *Effective compressive width per bar*

1.9.6.1 For running bond masonry, and masonry in other than running bond with bond beams spaced not more than 48 in. (1219 mm) center-to-center, the width of the compression area used to calculate element capacity shall not exceed the least of:

(a) Center-to-center bar spacing.

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

(c) 72 in. (1829 mm).

1.9.6.2 For masonry in other than running bond, with bond beams spaced more than 48 in. (1219 mm) center-to-center, the width of the compression area used to calculate element capacity shall not exceed the length of the masonry unit.

1.9.7 *Concentrated loads*

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

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

(b) The center-to-center distance between concentrated loads.

1.9.7.2 For walls laid in other than running bond, concentrated loads shall not be distributed across head joints. Where concentrated loads acting on such walls are applied to a bond beam, the

concentrated load is permitted to be distributed through the bond beam, but shall not be distributed across head joints below the bond beams.

1.10 — Deflection

1.10.1 *Deflection of beams and lintels*

~~Deflection of beams and lintels due to dead plus live loads shall not exceed the lesser of *l*/600 or 0.3 in. (7.6 mm) when providing vertical support to masonry designed in accordance with Section 2.2 or Chapter 5.~~ Masonry beams and lintels shall be designed to have adequate stiffness to limit deflections that adversely affect strength or serviceability.

1.10.1.1 Deflection of beams and lintels providing vertical support to masonry designed in accordance with Section 2.2, Section 3.2, Chapter 5, or Appendix Section A.2, shall not exceed the lesser of *l*/600 or 0.3 in. (7.6 mm).

1.10.1.2 Deflection of masonry beams and lintels shall be computed 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 computed with an effective moment of inertia, I_{eff}, as follows.

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

For continuous members, I_{eff} shall be permitted to be taken as the average of values obtained from Eq. (1-1) for the critical positive and negative moment regions. For members of uniform cross-section, I_{eff} shall be permitted to be taken as the value obtained from Eq. (1-1) at midspan for simple spans and at the support for cantilevers. For masonry designed in accordance with Chapter 2, the cracking moment, M_{cr}, shall be computed using the allowable flexural tensile stress taken from Table 2.2.3.2 multiplied by a factor of 2.5. For masonry designed in accordance with Chapter 3, the cracking moment ~~strength~~, M_{cr}, shall be computed using the value for the modulus of rupture, f_r, taken from Table 3.1.8.2.1. For masonry designed in accordance with Appendix A, the cracking moment ~~strength~~, M_{cr}, shall be computed using the value for the modulus of rupture, f_{rAAC}, as given by Section A.1.8.3.

1.10.1.3 Deflections of reinforced masonry beams and lintels 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 or lintel.

1.10.2 *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.

1.11 — Stack bond masonry

For masonry in other than running bond, the minimum area of horizontal reinforcement shall be 0.00028 ~~times~~ 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) ~~placed~~ on center in horizontal mortar joints or in bond beams ~~spaced not more than 48 in. (1219 mm) on center~~.

1.12 — Corbels

1.12.1 *Loadbearing corbels*

Loadbearing corbels shall be designed in accordance with Chapter 2, 3 or 4.

1.12.2 *Non-loadbearing corbels*

Non-loadbearing corbels shall be designed in accordance with Chapters 2, 3 or 4 or detailed as follows:

(a) Solid masonry units shall be used.

(b) The maximum projection beyond the face of the wall shall not exceed:

(1) one-half the wall thickness for multiwythe walls bonded by mortar or grout and wall ties or masonry headers, or

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

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

(1) one-half the nominal unit height.

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

(d) The back surface of the corbelled section shall remain within 1 in. (25.4 mm) of plane.

1.13 — Beams

Design of beams shall meet the requirements of Section 1.13 and Section 2.3.3.3, 3.3.4.2, or A.3.4.2.

1.13.1 *Span length*

Span lengths shall be in accordance with the following:

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

1.13.1.2 For determination of members, that are continuous over supports, span length shall be taken as the distance between centers of supports.

1.13.2 *Lateral support*

The compression face of beams shall be laterally supported at a maximum spacing of 32 multiplied by the beam thickness.

1.14 — Columns

Design of columns shall meet the requirements of ~~this section~~Section 1.14.1 or Section 1.14.2, and the requirements of Section 2.1.6, 3.3.4.4, or A.3.4.4.

1.14.1 *General column design*

1.14.1.1 *Dimensional limits* — Minimum side dimension shall be 8 in. (203 mm) nominal.

1.14.1.2 *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.

1.14.1.3 *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. Lateral ties shall be placed in either a mortar joint or in

grout. Where longitudinal bars are located around the perimeter of a circle, a complete circular lateral tie is permitted. Lap length for circular ties shall be 48 tie diameters.

(d) Lateral ties shall be located vertically not more than one-half lateral tie spacing above the top of footing or slab in any story, and shall be spaced not more than one-half a lateral tie spacing below the lowest horizontal reinforcement in beam, girder, slab, or drop panel above.

(e) Where beams or brackets frame into a column from four directions, lateral ties shall be permitted to be terminated not more than 3 in. (76.2 mm) below the lowest reinforcement in the shallowest of such beams or brackets.

1.14.2 *Lightly loaded columns* — Masonry columns used only to support light frame roofs of carports, porches, sheds or similar structures assigned to Seismic Design Category A, B, or C, which are subject to unfactored gravity loads not exceeding 2000 pounds (8900 N) 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 grouted solid.

~~1.13~~1.15 — Details of reinforcement and metal accessories

~~1.13~~1.15.1 *Embedment*

Reinforcing bars shall be embedded in grout.

~~1.13~~1.15.2 *Size of reinforcement*

~~1.13~~1.15.2.1 The maximum size of reinforcement used in masonry shall be No. 11 (M #36).

~~1.13~~1.15.2.2 The diameter of reinforcement shall not exceed one-half the least clear dimension of the cell, bond beam, or collar joint in which it is placed. ~~(See Section 1.16~~1.18.1.~~)~~

~~1.13~~1.15.2.3 Longitudinal and cross wires of joint reinforcement shall have a minimum wire size of W1.1 (MW7) and a maximum wire size of one-half the joint thickness.

~~1.13~~1.15.3 *Placement of reinforcement*

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

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

1.131.15.3.3 The clear distance limitations between bars required in Sections ~~1.13~~1.15.3.1 and ~~1.13~~1.15.3.2 shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

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

1.131.15.3.5 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.

1.131.15.4 *Protection of reinforcement*

1.131.15.4.1 Reinforcing bars 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 No. 5 (M #16) bars or smaller.

(b) Masonry not exposed to earth or weather: $1^1/_2$ in. (38.1 mm).

1.131.15.4.2 Longitudinal wires of joint reinforcement 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 shall be stainless steel or protected from corrosion by hot-dipped galvanized coating or epoxy coating when used in masonry exposed to earth or weather and in interior walls exposed to a mean relative humidity exceeding 75 percent. All other joint reinforcement shall be mill galvanized, hot-dip galvanized, or stainless steel.

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

~~1.13~~1.15.5 *Standard hooks*

Standard hooks shall be formed by one of the following methods:

(a) A 180-degree turn plus extension of at least 4 bar diameters but not less than $2^1/_2$ in. (64 mm) at free end of bar.

(b) A 90-degree turn plus extension of at least 12 bar diameters at free end of bar.

(c) For stirrup and tie anchorage only, either a 90-degree or a 135-degree turn plus an extension of at least 6 bar diameters at the free end of the bar.

~~1.13~~1.15.6 *Minimum bend diameter for reinforcing bars*

The diameter of bend measured on the inside of reinforcing bars, other than for stirrups and ties, shall not be less than values specified in Table ~~1.13~~1.15.6.

Table ~~1.13~~1.15.6 — Minimum diameters of bend

Bar size and type	Minimum diameter
No. 3 through No. 7 (M #10 through #22) Grade 40 (Grade 300)	5 bar diameters
No. 3 through No. 8 (M #10 through #25) Grade 50 or 60 (Grade 350 or 420)	6 bar diameters
No. 9, No. 10, and No. 11 (M #29, #32, and #36) Grade 50 or 60 (Grade 350 or 420)	8 bar diameters

1.16 — Anchor bolts

Headed and bent-bar anchor bolts shall conform to the provisions of Sections 1.16.1 through 1.16.7.

1.16.1 *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 1.16, 2.1.4 and 3.1.6, are permitted to be considered as if they are embedded in grout.

When anchor bolts are placed in fine grout, there shall be a minimum of ¼ in. (6.4 mm) of grout between bolts and masonry unit; when anchor bolts are placed in coarse grout, there shall be a minimum of ½ in. (12.7 mm) of grout between bolts and 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).

1.16.2 *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 Eq. (1-2).

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

The portion of projected area overlapping an open cell, or open head joint, or that lies outside the masonry shall be deducted from the value of A_{pt} calculated using Eq. (1-2). Where the projected areas of anchor bolts overlap, the value of A_{pt} calculated using Eq. (1-2) shall be adjusted so that no portion of masonry is included more than once.

1.16.3 *Projected area for shear* – The projected area of headed and bent-bar anchor bolts loaded in shear, A_{pv}, shall be determined from Eq. (1-3).

$$A_{pv} = \frac{\pi l_{be}^2}{2} \qquad (1\text{-}3)$$

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 Eq. (1-3). Where the projected areas of anchor bolts overlap, the value of A_{pv} calculated using Eq. (1-3) shall be adjusted so that no portion of masonry is included more than once.

1.16.4 *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.

1.16.5 *Effective embedment length of 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.

1.16.6 *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).

1.16.7 *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.

NOTE – All of Section 1.17 is new but is not shown underlined to try to highlight revisions made to the new text.

~~1.14~~1.16~~1~~1.17 — Seismic design requirements

~~**1.141.16.1** *Scope*~~

~~The seismic design requirements of this section apply to the design and construction of masonry, except glass unit masonry and masonry veneer.~~

~~**1.141.16.2** *General*~~

~~**1.141.16.2.1** *Seismic design category classification* — Masonry shall comply with the requirements of Sections 1.141.16.3 through 1.141.16.7 based on the Seismic Design Category as defined in ASCE 7-02. In addition, masonry shall comply with either the requirements of Section 1.1.3 or the requirements of Section 2.1.3.4.~~

~~**1.141.16.2.2** *Lateral force resisting system* — Buildings relying on masonry shear walls as part of the lateral force-resisting system shall have shear walls that comply with the requirements of Section 1.141.16.2.2.1, 1.141.16.2.2.2, 1.141.16.2.2.3, 1.141.16.2.2.4, or 1.141.16.2.2.5.~~

~~Exception: Buildings assigned to Seismic Design Category A shall be permitted to have shear walls complying with Section 5.3.~~

~~Exception: AAC masonry shear walls shall be permitted to comply with Sections 1.141.16.2.2.6, 1.141.16.2.2.7 or 1.141.16.2.2.8.~~

~~1.14~~1.16.2.2.1 ~~*Ordinary plain (unreinforced) masonry shear walls*~~ — ~~Design of ordinary plain (unreinforced) masonry shear walls shall comply with the requirements of Section 2.2, Section 3.2, or Chapter 4.~~

~~1.14~~1.16.2.2.2 ~~*Detailed plain (unreinforced) masonry shear walls*~~ — ~~Design of detailed plain (unreinforced) masonry shear walls shall comply with the requirements of Section 2.2 or Section 3.2, and shall comply with the requirements of Sections 1.14~~1.16.2.2.2.1 ~~and 1.14~~1.16.2.2.2.2.

~~1.14~~1.16.2.2.2.1 ~~*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.~~

~~Reinforcement adjacent to openings need not be provided for openings smaller than 16 in. (406 mm) in either the horizontal or vertical direction, unless the spacing of distributed reinforcement is interrupted by such openings.~~

~~Horizontal joint reinforcement shall consist of at least two wires of W1.7 (MW11) spaced not more than 16 in. (406 mm) on center, or bond beam reinforcement shall be provided of at least 0.2 in.^{2} (129 mm^2) in cross-sectional area spaced not more than 120 in. (3048 mm) on center. Horizontal reinforcement shall also be provided at the bottom and top of wall openings and shall extend not less than 24 in. (610 mm) nor less than 40 bar diameters past the opening, continuously at structurally connected roof and floor levels, and within 16 in. (406 mm) of the top of walls.~~

~~1.14~~1.16.2.2.2.2 ~~*Connections*~~ — ~~Connectors shall be provided to transfer forces between masonry walls and horizontal elements in accordance with the requirements of Section 2.1.8. Connectors shall be designed to transfer horizontal design forces acting either perpendicular or parallel to the wall, but not less than 200 lb per lineal ft (2919 N per lineal m) of wall. The maximum spacing between connectors shall be 4 ft (1.22 m).~~

~~1.14~~1.16.2.2.3 ~~*Ordinary reinforced masonry shear walls*~~ — ~~Design of ordinary reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.3, and shall comply with the requirements of Sections 1.14~~1.16.2.2.2.1 ~~and 1.14~~1.16.2.2.2.2.

~~1.14~~1.16.2.2.4 ~~*Intermediate reinforced masonry shear walls*~~ — ~~Design of intermediate reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.3.~~

Design shall also comply with the requirements of Sections 1.141.16.2.2.2.1 and 1.141.16.2.2.2.2, except that the spacing of vertical reinforcement shall not exceed 48 in. (1219 mm).

1.141.16.2.2.5 *Special reinforced masonry shear walls* — Design of special reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.3. Design shall also comply with the requirements of Sections 1.141.16.2.2.2.1, 1.141.16.2.2.2.2, 1.141.16.6.3, and the following:

(a)The maximum spacing of vertical and horizontal reinforcement shall be the smaller of one-third the length of the shear wall, one-third the height of the shear wall, or 48 in. (1219 mm).

(b)The minimum cross-sectional area of vertical reinforcement shall be one-third of the required shear reinforcement.

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

1.141.16.2.2.6 *Ordinary plain (unreinforced) AAC masonry shear walls* — Design of ordinary plain (unreinforced) AAC masonry shear walls shall comply with the requirements of Section A.2.

1.141.16.2.2.7 *Detailed plain (unreinforced) AAC masonry shear walls* — Design of detailed plain (unreinforced) AAC masonry shear walls shall comply with the requirements of Section A.2 and the minimum reinforcement requirements for members of the lateral force-resisting system in AAC masonry as follows: 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. Reinforcement adjacent to openings need not be provided for openings smaller than 16 in. (406 mm), unless the minimum reinforcement requirements are interrupted by such openings. Horizontal reinforcement shall be provided at the bottom and top of wall openings and shall extend not less than 24 in. (610 mm) nor less than 40 bar diameters past the opening.

1.141.16.2.2.8 *Ordinary reinforced AAC masonry shear walls* — Design of ordinary reinforced AAC masonry shear walls shall comply with the requirements of Section A.3 and the requirements of Section 1.141.16.2.2.7.

1.141.16.3 *Seismic Design Category A*

1.141.16.3.1 — Structures in Seismic Design Category A shall comply with the requirements of Chapter 2, 3, 4, or 5. AAC masonry structures in Seismic Design Category A shall comply with the requirements of Appendix A.

1.141.16.3.2 *Drift limits* — The calculated story drift of masonry structures due to the combination of design seismic forces and gravity loads shall not exceed 0.007 times multiplied by the story height.

1.141.16.3.3 *Anchorage of masonry walls* — Masonry walls shall be anchored to the roof and all floors that provide lateral support for the walls. The anchorage shall provide a direct connection between the walls and the floor or roof construction. The connections shall be capable of resisting the greater of a seismic lateral force induced by the wall or 1000 times multiplied by the effective peak velocity-related acceleration, lb per lineal ft of wall (14,590 times multiplied by, N/m).

Exception: AAC masonry walls shall comply with the requirements of Section 1.141.16.4.3.

1.141.16.4 *Seismic Design Category B*

1.141.16.4.1 Structures in Seismic Design Category B shall comply with the requirements of Seismic Design Category A and with the additional requirements of Section 1.141.16.4. AAC masonry structures shall comply with the requirements of 1.141.16.4.3.

1.141.16.4.2 *Design of elements that are part of the lateral force resisting system* — The lateral force resisting system shall be designed to comply with the requirements of Chapter 2, 3, or 4. Masonry shear walls shall comply with the requirements of ordinary plain (unreinforced) masonry shear walls, detailed plain (unreinforced) masonry shear walls, ordinary reinforced masonry shear walls, intermediate reinforced masonry shear walls, or special reinforced masonry shear walls.

1.141.16.4.3 *Anchorage of floor and roof diaphragms in AAC masonry structures* — Floor and roof diaphragms in AAC masonry structures shall be surrounded by a continuous grouted bond beam reinforced with at least two longitudinal reinforcing bars, having a total cross-sectional area of at least 0.4 in.^{2} (260 mm^2).

1.141.16.5 *Seismic Design Category C*

1.141.16.5.1 Structures in Seismic Design Category C shall comply with the requirements of Seismic Design Category B and with the additional requirements of Section 1.141.16.5.

1.141.16.5.2 *Design of elements that are not part of the lateral force resisting system*

~~**1.141.16.5.2.1** Load-bearing frames or columns that are not part of the lateral force-resisting system shall be analyzed as to their effect on the response of the system. Such frames or columns shall be adequate for vertical load carrying capacity and induced moment due to the design story drift.~~

~~**1.141.16.5.2.2** Masonry partition walls, masonry screen walls and other masonry elements that are not designed to resist vertical or lateral loads, other than those induced by their own mass, shall be isolated from the structure so that vertical and lateral forces are not imparted to these elements. Isolation joints and connectors between these elements and the structure shall be designed to accommodate the design story drift.~~

~~**1.141.16.5.2.3** *Reinforcement requirements* — Masonry elements listed in Section 1.141.16.5.2.2, except AAC masonry elements, shall be reinforced in either the horizontal or vertical direction in accordance with the following:~~

~~(a) *Horizontal reinforcement* — Horizontal joint reinforcement shall consist of at least two longitudinal W1.7 (MW11) wires spaced not more than 16 in. (406 mm) for walls greater than 4 in. (102 mm) in width and at least one longitudinal W1.7 (MW11) wire spaced not more 16 in. (406 mm) for walls not exceeding 4 in. (102 mm) in width; or at least one No. 4 (M #13) bar spaced not more than 48 in. (1219 mm). Where two longitudinal wires of joint reinforcement are used, the space between these wires shall be the widest that the mortar joint will accommodate. Horizontal reinforcement shall be provided within 16 in. (406 mm) of the top and bottom of these masonry walls.~~

~~(b) *Vertical reinforcement* — Vertical reinforcement shall consist of at least one No. 4 (M #13) bar spaced not more than 120 in. (3048 mm) for Seismic Design Category C and not more than 48 in. (1219 mm) for Seismic Design Category D, E, and F. Vertical reinforcement shall be located within 16 in. (406 mm) of the ends of masonry walls.~~

~~**1.141.16.5.3** *Design of elements that are part of the lateral force-resisting system* — Design of masonry columns and shear walls shall comply with the requirements of 1.141.16.5.3.1 and 1.141.16.5.3.2. Design of ordinary reinforced AAC masonry structures shall comply with the requirements of 1.141.16.5.3.3.~~

~~**1.141.16.5.3.1** *Connections to masonry columns* — Connectors shall be provided to transfer forces between masonry columns and horizontal elements in accordance with the requirements of Section 2.1.8. 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.

1.141.16.5.3.2 *Masonry shear walls* — Masonry shear walls shall comply with the requirements for ordinary reinforced masonry shear walls, intermediate reinforced masonry shear walls, or special reinforced masonry shear walls.

1.141.16.5.3.3 *Anchorage of floor and roof diaphragms in AAC masonry structures* — Lateral load between floor and roof diaphragms and AAC masonry shear walls shall be transferred through connectors embedded in grout in accordance with Section 2.1.8. Connectors shall be designed to transfer horizontal design forces acting either parallel or perpendicular to the wall but not less than 200 lb per lineal ft (2919 N per lineal m) of wall. The maximum spacing between connectors shall be 4 ft (1.2 m).

1.141.16.6 *Seismic Design Category D*

1.141.16.6.1 Structures in Seismic Design Category D shall comply with the requirements of Seismic Design Category C and with the additional requirements of Section 1.141.16.6.

Exception: AAC masonry elements shall comply with the requirements of 1.141.16.5.

1.141.16.6.2 *Design requirements* — Masonry elements, other than those covered by Section 1.141.16.5.2.2, shall be designed in accordance with the requirements of Sections 2.1 and 2.3, Chapter 3, Chapter 4 or Appendix A.

1.141.16.6.3 *Minimum reinforcement requirements for masonry walls* — Masonry walls other than those covered by Section 1.141.16.5.2.2, and other than AAC masonry, shall be reinforced in both the vertical and horizontal direction. The sum of the cross-sectional area of horizontal and vertical reinforcement shall be at least 0.002 times multiplied by the gross cross-sectional area of the wall, and the minimum cross-sectional area in each direction shall be not less than 0.0007 times multiplied by the gross cross-sectional area of the wall, using specified dimensions. Reinforcement shall be uniformly distributed. The maximum spacing of reinforcement shall be 48 in. (1219 mm), except for stack bond masonry. Wythes of stack bond masonry shall be constructed of fully grouted hollow open-end units, fully grouted hollow units laid with full head joints, or solid units. Maximum spacing of reinforcement for walls with stack bond masonry shall be 24 in. (610 mm).

~~**1.141.16.6.4** *Masonry shear walls* — Masonry shear walls shall comply with the requirements for special reinforced masonry shear walls.~~

~~**1.141.16.6.5** *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.~~

~~**1.141.16.6.6** *Material requirements* — Neither Type N mortar nor masonry cement shall be used as part of the lateral force-resisting system.~~

~~**1.141.16.6.7** *Lateral tie anchorage* — Standard hooks for lateral tie anchorage shall be either a 135-degree standard hook or a 180-degree standard hook.~~

~~**1.141.16.7** *Seismic Design Categories E and F*~~

~~**1.141.16.7.1** Structures in Seismic Design Categories E and F shall comply with the requirements of Seismic Design Category D and with the additional requirements of Section 1.141.16.7.~~

~~**1.141.16.7.2** *Minimum reinforcement for stack bond elements that are not part of the lateral force-resisting system* — Stack bond masonry that is not part of the lateral force-resisting system shall have a horizontal cross-sectional area of reinforcement of at least 0.0015 times multiplied by the gross cross-sectional area of masonry. The maximum spacing of horizontal reinforcement shall be 24 in. (610 mm). These elements shall be solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.~~

~~**1.141.16.7.3** *Minimum reinforcement for stack bond elements that are part of the lateral force-resisting system* — Stack bond masonry that is part of the lateral force-resisting system shall have a horizontal cross-sectional area of reinforcement of at least 0.0025 times multiplied by the gross cross-sectional area of masonry. The maximum spacing of horizontal reinforcement shall be 16 in. (406 mm). These elements shall be solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.~~

~~1.16~~1.17.1 *Scope*

The seismic design requirements of Section ~~1.16~~1.17 shall apply to the design and construction of masonry, except glass unit masonry and masonry veneer.

~~1.16~~1.17.2 *General analysis*

1.161.17.2.1 *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.

1.161.17.2.2 *Load path* – Structural masonry elements that transmit forces, resulting from seismic events, to the foundation shall comply with the requirements of Section 1.161.17.

1.161.17.2.3 *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 provide minimum load path connection requirements and anchorage design forces, the requirements of ASCE 7 shall be used.

1.161.17.2.4 *Drift limits* —Under loading combinations that include earthquake, masonry structures shall be designed so the calculated story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from the legally adopted building code. When the legally adopted building code does not provide allowable story drifts, structures shall be designed so the calculated story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from ASCE 7.

It shall be permitted to assume that the following shear wall types comply with the story drift limits of ASCE 7: empirical, ordinary plain (unreinforced), detailed plain (unreinforced), ordinary reinforced, intermediate reinforced, and ordinary plain (unreinforced) AAC masonry shear walls, and detailed plain (unreinforced) AAC masonry shear walls.

1.161.17.3 *Element classification* – Masonry elements shall be classified as either participating or nonparticipating elements of the seismic force-resisting system in accordance with Section 1.161.17.3.

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

~~1.16~~1.17.3.2 *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 ~~1.16~~1.17.3.2.1, ~~1.16~~1.17.3.2.2, ~~1.16~~1.17.3.2.3, ~~1.16~~1.17.3.2.4, ~~1.16~~1.17.3.2.5, ~~1.16~~1.17.3.2.6, ~~1.16~~1.17.3.2.7, ~~1.16~~1.17.3.2.8, ~~1.16~~1.17.3.2.9, ~~1.16~~1.17.3.2.10, 1.17.3.2.11 or 1.17.3.2.12.

~~1.16~~1.17.3.2.1 *Empirical design of masonry shear walls* – Empirical design of shear walls shall comply with the requirements of Section 5.3.

~~1.16~~1.17.3.2.2 *Ordinary plain (unreinforced) masonry shear walls* – Design of ordinary plain (unreinforced) masonry shear walls shall comply with the requirements of Section 2.2~~,~~ or Section 3.2~~, or Chapter 4~~.

1.17.3.2.3 *Detailed plain (unreinforced) masonry shear walls* – Design of detailed plain (unreinforced) masonry shear walls shall comply with the requirements of Section 2.2 or Section 3.2, and shall comply with the requirements of Section 1.17.3.2.3.1.

~~1.16~~1.17.3.2.3~~3~~.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.

Reinforcement adjacent to openings need not be provided for openings smaller than 16 in. (406 mm) in either the horizontal or vertical direction, unless the ~~spacing of~~ distributed reinforcement is interrupted by such openings.

Horizontal ~~joint~~ 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 bond beam reinforcement shall be provided~~or ~~of~~ 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 bottom and top of wall openings and shall extend not less than 24 in. (610 mm) nor less than 40 bar diameters past the opening, continuously at structurally connected roof and floor levels, and within 16 in. (406 mm) of the top of walls.

~~1.16~~1.17.3.2.~~3~~ 4 *Ordinary reinforced masonry shear walls* – Design of ordinary reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.3, and shall comply with the requirements of Section ~~1.16~~1.17.3.2.3.1.

~~1.16~~1.17.3.2.~~4~~ 5 *Intermediate reinforced masonry shear walls* – Design of intermediate reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.3. Reinforcement detailing shall also comply with the requirements of Section ~~1.16~~1.17.3.2.3.1, except that the spacing of vertical reinforcement shall not exceed 48 in. (1219 mm).

~~1.16~~1.17.3.2.~~5~~ 6 *Special reinforced masonry shear walls* – Design of special reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.3. Reinforcement detailing shall also comply with the requirements of Section ~~1.16~~1.17.3.2.3.1 and the following:

(a) The maximum spacing of vertical reinforcement shall be the smallest of one-third the length of the shear wall, one-third the height of the shear wall, and 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry laid in other than running bond.

(b) The maximum spacing of horizontal reinforcement required to resist in-plane shear shall be uniformly distributed, shall be the smaller of one-third the length of the shear wall and one-third the height of the shear wall, and shall be embedded in grout. The maximum spacing of horizontal reinforcement shall not exceed 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry laid in other than running bond.

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

1. For masonry laid in running bond, the minimum cross-sectional area of reinforcement in each direction shall be not less than 0.0007 multiplied by the gross cross-sectional area of the wall, using specified dimensions.

2. For masonry laid in other than running bond, the minimum cross-sectional area of vertical reinforcement shall be not less than 0.0007 multiplied by the gross cross-sectional area of the wall, using specified dimensions. The minimum cross-sectional area of horizontal reinforcement shall be not less than 0.0025 multiplied by the gross cross-sectional area of the wall, using specified dimensions.

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

(e) Masonry laid in other than running bond shall be solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.

~~1.16~~1.17.3.2.~~6~~ 7 *Ordinary plain (unreinforced) AAC masonry shear walls* – Design of ordinary plain (unreinforced) AAC masonry shear walls shall comply with the requirements of Appendix A.2 and Section ~~1.16~~1.17.3.2.~~6~~7.1.

~~1.16~~1.17.3.2.~~6~~7.1 *Anchorage of floor and roof diaphragms in AAC masonry structures* — Floor and roof diaphragms in AAC masonry structures shall be anchored to ~~surrounded by~~ a continuous grouted bond beam reinforced with at least two longitudinal reinforcing bars, having a total cross-sectional area of at least 0.4 in.^{2} (260 mm^2).

1.17.3.2.8 *Detailed plain (unreinforced) AAC masonry shear walls* – Design of detailed plain (unreinforced) AAC masonry shear walls shall comply with the requirements of Appendix A.2 and Sections 1.17.3.2.7.1 and 1.17.3.2.8.1.

1.17.3.2.8.1 *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. Reinforcement adjacent to openings need not be provided for openings smaller than 16 in. (406 mm), unless the minimum reinforcement requirements are interrupted by such openings. Horizontal reinforcement shall be provided at the bottom and top of wall openings and shall extend not less than 24 in. (610 mm) nor less than 40 bar diameters past the opening.

1.161.17.3.2.7 9 *Ordinary reinforced AAC masonry shear walls* – Design of ordinary reinforced AAC masonry shear walls shall comply with the requirements of Appendix A.3 and Sections 1.161.17.3.2.67.1 and 1.161.17.3.2.78.1.

1.161.17.3.2.8 10 *Ordinary plain (unreinforced) prestressed masonry shear walls* – Design of ordinary plain (unreinforced) prestressed masonry shear walls shall comply with the requirements of Chapter 4.

1.161.17.3.2.9 11 *Intermediate reinforced prestressed masonry shear walls* – Intermediate reinforced prestressed masonry shear walls shall comply with the requirements of Chapter 4, the reinforcement detailing requirements of Section 1.161.17.3.2.3.1, and the following:

(a) Reinforcement shall be provided in accordance with Sections 1.161.17.3.2.56(a) and 1.161.17.3.2.56(b).

(b) The minimum area of ~~transverse~~ horizontal reinforcement shall be $0.0007bd_v$.

(c) Flexural elements subjected to load reversals shall be symmetrically reinforced.

(d) The nominal moment strength at any section along an element 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 1.161.17.3.2.3.1, 1.161.17.3.2.56(a), and 1.161.17.3.2.6 5(b).

(f) Tendons shall be located in cells that are grouted the full height of the wall.

1.161.17.3.2.10 12 *Special reinforced prestressed masonry shear walls* – Special reinforced prestressed masonry shear walls shall comply with the requirements of Chapter 4, the reinforcement detailing requirements of Sections 1.161.17.3.2.3.1 and 1.161.17.3.2.9 11 and the following:

(a) The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Sections 1.161.17.3.2.3.1 and 1.161.17.3.2.911.

(b) Prestressing tendons shall consist of bars conforming to ASTM A 722.

(c) All cells of the masonry wall shall be grouted.

(d) The requirements of Section 3.3.3.5 or 3.3.6.5 shall be met.

~~1.16~~1.17.4 *Seismic Design Category ~~classification~~ requirements* — The design of masonry elements shall comply with the requirements of Sections ~~1.16~~1.17.4.1 through ~~1.16~~1.17.4.5 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 7 shall be used.

~~1.16~~1.17.4.1 *Seismic Design Category A requirements* – Masonry elements in structures assigned to Seismic Design Category A shall comply with the requirements of Sections ~~1.16~~1.17.1, ~~1.16~~1.17.2, and ~~1.16~~1.17.4.1.

~~1.16~~1.17.4.1.1 *Design of nonparticipating elements* – Nonparticipating masonry elements shall comply with the requirements of Section ~~1.16~~1.17.3.1 and Chapter 2, 3, 4, or 5 or Appendix A.

~~1.16~~1.17.4.1.2 *Design of participating elements* – Participating masonry elements shall be designed to comply with the requirements of Chapter 2, 3, 4, or 5 or Appendix A. Masonry shear walls shall be designed to comply with the requirements of Section ~~1.16~~1.17.3.2.1, ~~1.16~~1.17.3.2.2, ~~1.16~~1.17.3.2.3, ~~1.16~~1.17.3.2.4, ~~1.16~~1.17.3.2.5, ~~1.16~~1.17.3.2.6, ~~1.16~~1.17.3.2.7, ~~1.16~~1.17.3.2.8, ~~1.16~~1.17.3.2.9, ~~or 1.16~~1.17.3.2.10, 1.17.3.2.11, or 1.17.3.2.12.

~~1.16~~1.17.4.2 *Seismic Design Category B requirements* – Masonry elements in structures assigned to Seismic Design Category B shall comply with the requirements of Seismic Design Category A and with the additional requirements of Section ~~1.16~~1.17.4.2.

~~1.16~~1.17.4.2.1 *Design of participating elements* – Participating masonry elements shall be designed to comply with the requirements of Chapter 2, 3, or 4 or Appendix A. Masonry shear walls shall be designed to comply with the requirements of Section ~~1.16~~1.17.3.2.2, ~~1.16~~1.17.3.2.3, ~~1.16~~1.17.3.2.4, ~~1.16~~1.17.3.2.5, ~~1.16~~1.17.3.2.6, ~~1.16~~1.17.3.2.7, ~~1.16~~1.17.3.2.8, ~~1.16~~1.17.3.2.9, or ~~1.16~~1.17.3.2.10, 1.17.3.2.11, or 1.17.3.2.12.

1.161.17.4.3 *Seismic Design Category C requirements* – Masonry elements in structures assigned to Seismic Design Category C shall comply with the requirements of Seismic Design Category B and with the additional requirements of Section 1.161.17.4.3.

1.161.17.4.3.1 *Design of nonparticipating elements* – Nonparticipating masonry elements shall comply with the requirements of Section 1.161.17.3.1 and Chapter 2, 3, or 4 or Appendix A. 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 joint reinforcement shall consist of at least two longitudinal wires of W1.7 (MW11) wires bed joint reinforcement spaced not more than 16 in. (406 mm) on center for walls greater than 4 in. (102 mm) in width and at least one longitudinal W1.7 (MW11) wire spaced not more 16 in. (406 mm) on center for walls not exceeding 4 in. (102 mm) in width; or at least one No. 4 (M #13) bar spaced not more than 48 in. (1219 mm) on center. Where two longitudinal wires of joint reinforcement are used, the space between these wires shall be the widest that the mortar joint will accommodate. Horizontal reinforcement shall be provided within 16 in. (406 mm) of the top and bottom of these masonry walls.

(b) *Vertical reinforcement* — Vertical reinforcement shall consist of at least one No. 4 (M #13) bar spaced not more than 120 in. (3048 mm). Vertical reinforcement shall be located within 16 in. (406 mm) of the ends of masonry walls.

1.161.17.4.3.2 *Design of participating elements* – Participating masonry elements shall be designed to comply with the requirements of Section 2.3, 3.3, or A.3. Masonry shear walls shall be designed to comply with the requirements of Section 1.16.3.2.3, 1.161.17.3.2.4, 1.161.17.3.2.5, 1.161.17.3.2.67, 1.161.17.3.2.9, and 1.16.3.2.101.17.3.2.11, or 1.17.3.2.12.

1.161.17.4.3.2.1 *Connections to masonry columns* — Connections shall be designed to transfer forces between masonry columns and horizontal elements in accordance with the requirements of Section 2.1.8. 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.

~~1.16~~1.17.4.3.2.2 *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 2.1.8.

~~1.16~~1.17.4.3.2.3 *Material requirements* – ASTM C 34, structural clay loadbearing wall tiles, shall not be used as part of the seismic force-resisting system.

~~1.16.4.3.2.4 *Shear capacity design*~~

~~1.16.4.3.2.4.1 When designing participating reinforced elements in accordance with Section 3.3 or A.3 or Chapter 4, the design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the element, except that the nominal shear strength, V_n, need not exceed 2.5 times required shear strength, V_u.~~

~~1.16.4.3.2.4.2 When designing participating reinforced elements in accordance with Section 2.3, the shear or diagonal tension stress resulting from in-plane seismic forces shall be increased by a factor of 1.5. The 1.5 multiplier need not be applied to the overturning moment.~~

~~1.16~~1.17.4.3.2.~~5~~ 4 *Lateral stiffness* — At each story level, at least 80 percent of the lateral stiffness shall be provided by lateral-force-resisting walls. Along each line of lateral resistance at a particular story level, at least 80 percent of the lateral stiffness shall be provided by lateral-force-resisting walls. Where seismic loads are determined based on a seismic response modification factor, *R*, not greater than 1.5, piers and columns ~~are~~ shall be permitted to be used to provide seismic load resistance.

~~1.16~~1.17.4.3.2.~~6~~ 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.

1.161.17.4.4 *Seismic Design Category D requirements* – Masonry elements in structures assigned to Seismic Design Category D shall comply with the requirements of Seismic Design Category C and with the additional requirements of Section 1.161.17.4.4.

Exception: Design of participating elements of AAC masonry shall comply with the requirements of 1.161.17.4.3.

1.161.17.4.4.1 *Minimum reinforcement requirements for nonparticipating elements* – Nonparticipating masonry walls and piers, 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 1.161.17.4.3.1(a).

(b) *Vertical reinforcement* — Vertical reinforcement shall consist of at least one No. 4 (M #13) bar spaced not more than 48 in. (1219 mm). Vertical reinforcement shall be located within 16 in. (406 mm) of the ends of masonry walls.

1.161.17.4.4.2 *Design of participating elements* – Masonry shear walls shall be designed to comply with the requirements of Section 1.161.17.3.2.56, 1.161.17.3.2.79, or 1.16.3.2.10or 1.17.3.2.12.

1.161.17.4.4.2.1 *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.

1.161.17.4.4.2.2 *Material requirements* — Neither Type N mortar nor masonry cement mortar shall be used to construct participating elements.

~~1.16~~1.17.4.4.2.3 *Lateral tie anchorage* — Standard hooks for lateral tie anchorage shall be either a 135- degree standard hook or a 180-degree standard hook.

1.17.4.4.2.4 *Shear capacity design*

1.17.4.4.2.4.1 When designing participating reinforced elements in accordance with Section 3.3 or A.3 or Chapter 4, the design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the element, except that the nominal shear strength, V_n, need not exceed 2.5 times required shear strength, V_u.

1.17.4.4.2.4.2 When designing participating reinforced elements in accordance with Section 2.3, the shear or diagonal tension stress resulting from in-plane seismic forces shall be increased by a factor of 1.5. The 1.5 multiplier need not be applied to the overturning moment.

~~1.16~~1.17.4.5 *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 Seismic Design Category D and with the additional requirements of Section ~~1.16~~1.17.4.5.

~~1.16~~1.17.4.5.1 *Minimum reinforcement for nonparticipating masonry elements laid in other than running bond* – Masonry laid in other than 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 solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.

~~1.15~~1.17~~1~~1.18 — Quality assurance program

The quality assurance program shall comply with the requirements of this section, depending on the facility function, as defined in the legally adopted building code or ASCE 7~~-02~~. The quality assurance program shall itemize the ~~methods used to verify~~requirements for verifying conformance of material composition, quality, storage, handling, preparation, and placement with the requirements of ACI 530.1/ASCE 6/TMS 602.

1.151.171.18.1 The minimum quality assurance program for masonry in non-essential facilities and designed in accordance with Chapter 5, 6, or 7 shall comply with Table 1.151.171.18.1.

1.151.171.18.2 The minimum quality assurance program for masonry in essential facilities and designed in accordance with Chapter 5, 6, or 7 shall comply with Table 1.151.171.18.2.

1.151.171.18.3 The minimum quality assurance program for masonry in non-essential facilities and designed in accordance with chapters other than Chapter 5, 6, or 7 shall comply with Table 1.151.171.18.2.

1.151.171.18.4 The minimum quality assurance program for masonry in essential facilities and designed in accordance with chapters other than Chapter 5, 6, or 7 shall comply with Table 1.151.171.18.3.

1.151.171.18.5 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.

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

Table 1.151.171.18.1 — Level A Quality Assurance

MINIMUM TESTS AND SUBMITTALS	MINIMUM INSPECTION
Certificates for materials used in masonry construction indicating compliance with the contract documents	Verify compliance with the approved submittals

MINIMUM TESTS AND SUBMITTALS
Certificates for materials used in masonry construction indicating compliance with the contract documents
MINIMUM INSPECTION
Verify compliance with the approved submittals

Table ~~1.15~~1.~~17~~1.18.2 — Level B Quality Assurance

MINIMUM TESTS AND SUBMITTALS		
Certificates for materials used in masonry construction indicating compliance ~~with the contract documents~~		
Verification of Slump flow and VSI as delivered to the site in accordance with Article 1.5.B.1.b.3) for self-consolidating grout		
Verification of f'_m and f'_{AAC} prior to construction, except where specifically exempted by this Code		
MINIMUM INSPECTION		
Inspection Task	**Frequency [(a)]**	
	Continuous	Periodic
1. Verify compliance with ~~the required inspection provisions of the contract documents and~~ the approved submittals		X
2. As masonry construction begins, verify that the following are in compliance:		
a. Proportions of site-prepared mortar		X
b. Construction or mortar joints		X
c. Grade and size of prestressing tendons and anchorages		X
d. Location of reinforcement, connectors, and prestressing tendons and anchorages		X
e. Prestressing technique		X
3. Prior to grouting, verify that the following are in compliance:		
a. Grout space		X
b. Grade, type, and size of reinforcement and anchor bolts, and prestressing tendons, and anchorages		X
c. Placement of reinforcement, connectors, and prestressing tendons and anchorages		X
d. Proportions of site-prepared grout and prestressing grout for bonded tendons		X
e. Construction of mortar joints		X
4. Verify during construction:		
a. Size and location of structural elements		X
b. Type, size, and location of anchors, including other details of anchorage of masonry to structural members, frames, or other construction		X
c. Welding of reinforcement	X	
d. Preparation, construction, and protection of masonry during cold weather (temperature below 40°F (4.4°C)) or hot weather (temperature ~~below~~above 90°F (32.2°C))		X
e. Application and measurement of prestressing force	X	
f. Placement of grout and prestressing grout for bonded tendons is in compliance	X	
5. Observe preparation of grout specimens, mortar specimens, and/or prisms		X

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

1

MINIMUM TESTS AND SUBMITTALS	MINIMUM INSPECTION
Certificates for materials used in masonry construction indicating compliance with the contract documents Verification of f'_m and f'_{AAC} prior to construction, except where specifically exempted by this Code	As masonry construction begins, verify that the following are in compliance: • proportions of site-prepared mortar • construction of mortar joints • location of reinforcement, connectors, and prestressing tendons and anchorages • prestressing technique Prior to grouting, verify that the following are in compliance: • grout space • grade and size of reinforcement, prestressing tendons, and anchorages • placement of reinforcement, connectors, and prestressing tendons and anchorages • proportions of site-prepared grout and prestressing grout for bonded tendons • construction of mortar joints Verify that the placement of grout and prestressing grout for bonded tendons is in compliance Observe preparation of grout specimens, mortar specimens, and/or prisms Verify compliance with the required inspection provisions of the contract documents and the approved submittals

Table ~~1.15~~1.~~17~~1.18.3 — Level C Quality Assurance

MINIMUM TESTS AND SUBMITTALS		
Certificates for materials used in masonry construction indicating compliance ~~with the contract documents~~		
Verification of f'_m and f'_{AAC} in accordance with Article 1.4C • ~~-~~prior to construction and for • ~~-~~every 5000 sq. ft (464.5 m^2) during construction		
Verification of proportions of materials in premixed or preblended mortar, ~~grout, and~~ prestressing grout, and grout other than self-consolidating grout, as delivered to the site		
Verification of Slump flow and VSI as delivered to the site in accordance with Article 1.5.B.1.b.3) for self-consolidating grout		
MINIMUM INSPECTION		
Inspection Task	**Frequency [(a)]**	
	Continuous	Periodic
1. Verify compliance with ~~the required inspection provisions of the contract documents and~~ the approved submittals		X
2. Verify that the following are in compliance:		
a. Proportions of site-prepared mortar		X
b. Grade, type, and size of reinforcement and anchor bolts, and prestressing tendons and anchorages		X
c. Placement of masonry units and construction of mortar joints		X
d. Placement of reinforcement, connectors, and prestressing tendons and anchorages	X	
e. Grout space prior to grouting	X	
f. Placement of grout and prestressing grout for bonded tendons	X	
g. Size and location of structural elements		X
h. Type, size, and location of anchors including other details of anchorage of masonry to structural members, frames, or other construction	X	
i. Welding of reinforcement	X	
j. Preparation, construction, and protection of masonry during cold weather (temperature below 40°F (4.4°C)) or hot weather (temperature ~~below~~above 90°F (32.2°C))		X
k. Application and measurement of prestressing force	X	
3. Observe preparation of grout specimens, mortar specimens, and/or prisms	X	

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

~~MINIMUM TESTS AND SUBMITTALS~~	~~MINIMUM INSPECTION~~
~~Certificates for materials used in masonry construction indicating compliance with the contract documents.~~ ~~Verification of f'_m and f'_{AAC}:~~ ~~• prior to construction~~ ~~• every 5000 sq. ft (464.5 m^2) during construction~~ ~~Verification of proportions of materials in premixed or preblended mortar, grout, and prestressing grout as delivered to the site~~	~~From the beginning of masonry construction and continuously during construction of masonry:~~ ~~a)verify that the following are in compliance:~~ ~~• proportions of site-mixed mortar, grout, and prestressing grout for bonded tendons~~ ~~• grade and size of reinforcement, prestressing tendons and anchorages~~ ~~• placement of masonry units and construction of mortar joints~~ ~~• placement of reinforcement, connectors, and prestressing tendons and anchorages~~ ~~• grout space prior to grouting~~

	• placement of grout and prestressing grout for bonded tendons b) Observe preparation of grout specimens, mortar specimens, and/or prisms c) Verify compliance with the required inspection provisions of the contract documents and the approved submittals

1.151.171.18.7 *Acceptance relative to strength requirements*

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

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

1.161.181.19 — Construction

1.161.181.19.1 *Grouting, minimum spaces*

The minimum dimensions of spaces provided for the placement of grout shall be in accordance with Table 1.161.181.19.1. Higher gGrout pours with heights exceeding those shown in Table 1.19.1, higher grout lifts, smaller cavity widths, or smaller cell sizes smaller than those shown permitted in Table 1.161.181.19.1 or higher grout lift heights exceeding than those permitted by Article 3.5 D of ACI 530.1/ASCE 6/TMS 602 are permitted if the results of a grout demonstration panel show that the grout spaces are filled and adequately consolidated. In that case, the procedures used in constructing the grout demonstration panel shall be the minimum acceptable standard for grouting, and the quality assurance program shall include inspection during construction to verify grout placement.

1.16.21.181.19.2 *Embedded conduits, pipes, and sleeves*

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

1.16.21.181.19.2.1 Design shall not consider conduits, pipes, or sleeves as structurally replacing the displaced masonry.

1.16.21.181.19.2.2 Design shall consider the structural effects resulting from the removal of masonry to allow for the placement of pipes or conduits.

~~1.16.2~~1.18~~1~~1.19.2.3 Conduits, pipes, and sleeves in masonry shall be no closer than 3 diameters on center.

~~1.16.2~~1.18~~1~~1.19.2.4 Vertical conduits, pipes, or sleeves placed in masonry columns or pilasters shall not displace more than 2 percent of the net cross section.

~~1.16.2~~1.18~~1~~1.19.2.5 Pipes shall not be embedded in masonry when:

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

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

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

Table ~~1.16~~1.18~~9~~.1 — Grout space requirements

Grout type[1]	Maximum grout pour height, ft (m)	Minimum width of grout space,[2,3] in. (mm)	Minimum 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 (1.52)	2 (50.8)	2 x 3 (50.8 x 76.2)
Fine	12 (3.66)	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 (1.52)	2 (50.8)	2 1/2 x 3 (63.5 x 76.2)
Coarse	12 (3.66)	2 1/2 (63.5)	3 x 3 (76.2 x 76.2)
Coarse	24 (7.32)	3 (76.2)	3 x 4 (76 .2x 102)

[1] Fine and coarse grouts are defined in ASTM C 476.

[2] For grouting between masonry wythes.

[3] Grout space dimension is the clear dimension between any masonry protrusion and shall be increased by the diameters of the horizontal bars within the cross section of the grout space.

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

~~1.19 — Concentrated loads~~

~~1.19.1 Concentrated loads shall not be distributed over a length greater than the minimum of the following:~~

~~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.~~

~~The center-to-center distance between concentrated loads.~~

~~**1.19.2** For walls laid in other than running bond, concentrated loads shall not be distributed across head joints. Where concentrated loads acting on such walls are applied to a bond beam, the concentrated load is permitted to be distributed through the bond beam, but shall not be distributed across head joints below the bond beams.~~

CHAPTER 2
ALLOWABLE STRESS DESIGN OF MASONRY

2.1 — General

2.1.1 *Scope*

This chapter provides requirements for allowable stress design of masonry. Masonry design in accordance with this chapter shall comply with the requirements of Chapter 1, this section, and either Section 2.2 or 2.3.

2.1.2 *Load combinations*

2.1.2.1 When the legally adopted building code does not provide load combinations, structures and members shall be designed to resist the most restrictive of the following combination of loads:

(a) D

(b) $D + L$

(c) $D + L +$ (W or E)

(d) $D + W$

(e) $0.9\,D + E$

(f) $D + L +$ (H or F)

(g) $D +$ (H or F)

(h) $D + L + T$

(i) $D + T$

2.1.2.2 For prestressed masonry members, the prestressing force shall be added to ~~all~~ load combinations.

2.1.2.3 Unless prohibited by the legally adopted building code, ~~The~~ allowable stresses and allowable loads in Chapters 2 and 4 shall be permitted to be increased by one-third when considering Load Combination (c), (d), or (e) of Section 2.1.2.1~~, and as permitted by the legally adopted building code~~.

2.1.3 *Design strength*

2.1.3.1 Project drawings shall show the specified compressive strength of masonry, f'_m, for each part of the structure.

2.1.3.2 Each portion of the structure shall be designed based on the specified compressive strength of masonry, f'_m, for that part of the work.

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

~~**2.1.3.4** *Strength requirements* For masonry structures designed using loading combinations for strength design and not designed in accordance with Chapter 3, the provisions of this section shall apply. The design strength of masonry structures and masonry elements shall be at least equal to the required strength determined in accordance with this section, except for masonry structures and masonry elements in Seismic Design Category A designed in accordance with the provisions of Chapter 5.~~

~~**2.1.3.4.1** *Required strength* — Required strength, U, to resist seismic forces in combination with gravity and other loads, including load factors, shall be as required in the earthquake loads section of ASCE 7 - 02. For prestressed masonry, the response modification factor (R) and the deflection amplification factor (C_d), indicated in ASCE 7-02 Table 9.5.2.2 for ordinary plain (unreinforced) masonry shear walls, shall be used in determining the base shear and design story drift.~~

~~**2.1.3.4.2** *Nominal strength* The nominal strength of masonry shall be taken as $2^{1}/_{2}$ times the allowable stress value, except as allowed in Section 4.1.5 for prestressed masonry. The allowable stress values shall be determined in accordance with Chapter 2 or 4 and are permitted to be increased by one-third for load combinations including earthquake.~~

~~**2.1.3.4.3** *Design strength* — The design strength of masonry provided by a member, its connections to other members, and its cross sections in terms of flexure, axial load, and shear shall be taken as the nominal strength multiplied by a strength reduction factor, ϕ, as follows:~~

~~(a) Axial load and flexure except for flexural tension in unreinforced masonry $\phi = 0.8$~~

~~(b) Flexural tension in unreinforced masonry $\phi = 0.4$~~

~~(c) Shear ..... $\phi = 0.6$~~

~~(d) Shear and tension on anchor bolts embedded in masonry ..... $\phi = 0.6$~~

2.1.4 *Anchor bolts embedded in grout*

2.1.4.1 *Design requirements* - Anchor bolts shall be designed using either the provisions of Section 2.1.4.2 or, for headed and bent-bar anchors, by the provisions of Section 2.1.4.3.

2.1.4.2 *Allowable loads determined by test*

2.1.4.2.1 Anchors shall be tested in accordance with ASTM E 488, except that a minimum of five tests shall be performed. Loading conditions of the test shall be representative of intended use of the anchor bolt.

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

2.1.4.3 *Allowable loads determined by calculation for headed and bent bar anchors* - Allowable loads for headed and bent-bar anchor bolts embedded in grout shall be determined in accordance with the provisions of Sections 2.1.4.3.1 through 2.1.4.3.3.

2.1.4.3.1 *Allowable axial tensile load of headed and bent-bar anchor bolts* - The allowable axial tensile load of headed anchor bolts shall be computed using the provisions of Sections 2.1.4.3.1.1. The allowable axial tensile load of bent-bar anchor bolts shall be computed using the provisions of Section 2.1.4.3.1.2.

2.1.4.3.1.1 *Allowable axial tensile load of headed anchor bolts* — The allowable axial tensile load, B_a, of headed anchor bolts embedded in grout shall be determined by Eq. (2-1) (allowable axial tensile load governed by masonry breakout) or Eq. (2-2) (allowable axial tensile load governed by steel yielding). The allowable axial tensile load, B_a, shall be the smaller of the values obtained from Eqs. (2-1) and (2-2).

$$B_{ab} = 1.25\, A_{pt} \sqrt{f'_m} \qquad (2\text{-}1)$$

$$B_{as} = 0.6 A_b f_y \qquad (2\text{-}2)$$

2.1.4.3.1.2 *Allowable axial tensile load of bent-bar anchor bolts* — The allowable axial tensile load, B_a, for bent-bar anchor bolts embedded in grout shall be determined by Eq. (2-3) (allowable axial tensile load governed by masonry breakout), Eq. (2-4) (allowable axial tensile load governed by anchor bolt pullout), or Eq. (2-5) (allowable axial tensile load governed by steel yielding). The allowable axial tensile load, B_a, shall be the smallest of the values obtained from Eqs. (2-3), (2-4) and (2-5).

$$B_{ab} = 1.25\, A_{pt} \sqrt{f'_m} \qquad (2\text{-}3)$$

$$B_{ap} = 0.6\, f'_m e_b\, d_b + 120\pi (l_b + e_b + d_b)\, d_b \qquad (2\text{-}4)$$

$$B_{as} = 0.6 A_b f_y \qquad (2\text{-}5)$$

2.1.4.3.2 *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 determined by Eq. (2-6) (allowable shear load governed by masonry breakout), Eq. (2-7) (allowable shear load governed by masonry crushing), Eq. (2-8) (allowable shear load governed by anchor bolt pryout) or Eq. (2-9) (allowable shear load governed by steel yielding). The allowable shear load, B_v, shall be the smallest of the values obtained from Eqs. (2-6), (2-7), (2-8) and (2-9).

$$B_{vb} = 1.25\, A_{pv} \sqrt{f'_m} \qquad (2\text{-}6)$$

$$B_{vc} = 350 \sqrt[4]{f'_m}\, A_b \qquad (2\text{-}7)$$

$$B_{vpry} = 2.0 B_{ab} = 2.5 A_{pt} \sqrt{f'_m} \qquad (2\text{-}8)$$

$$B_{vs} = 0.36 A_b f_y \qquad (2\text{-}9)$$

2.1.4.3.3 *Combined axial tension and shear* — Anchor bolts subjected to axial tension in combination with shear shall satisfy Eq. (2-10).

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \le 1 \qquad (2\text{-}10)$$

~~**2.1.4.1** *Test design requirements* — Except as provided in Section 2.1.4.2, anchor bolts shall be designed based on the following provisions.~~

~~**2.1.4.1.1** Anchors shall be tested in accordance with ASTM E 488 under stresses and conditions representing intended use, except that a minimum of five tests shall be performed.~~

~~**2.1.4.1.2** Allowable loads shall not exceed 20 percent of the average tested strength.~~

~~**2.1.4.2** *Plate, headed, and bent-bar anchor bolts* — The allowable loads for plate anchors, headed anchor bolts, and bent-bar anchor bolts (J- or L-type) embedded in masonry shall be determined in accordance with the provisions of Sections 2.1.4.2.1 through 2.1.4.2.4.~~

~~**2.1.4.2.1** The minimum effective embedment length shall be 4 bolt diameters, but not less than 2 in. (50.8 mm).~~

~~**2.1.4.2.2** The allowable load in tension shall be the lesser of that given by Eq. (2-1) or Eq. (2-2).~~

~~$$B_a = 0.5 A_p \sqrt{f'_m} \qquad (2\text{-}1)$$~~

~~$$B_a = 0.2 A_b f_y \qquad (2\text{-}2)$$~~

~~**2.1.4.2.2.1** The area A_p shall be the lesser of Eq. (2-3) or Eq. (2-4). Where the projected areas of adjacent anchor bolts overlap, A_p of each bolt shall be reduced by one-half of the overlapping area. That portion of the projected area falling in an open cell or core shall be deducted from the value of A_p calculated using Eq. (2-3) or (2-4).~~

~~$$A_p = \pi l_b^2 \qquad (2\text{-}3)$$~~

~~$$A_p = \pi l_{be}^2 \qquad (2\text{-}4)$$~~

~~**2.1.4.2.2.2** The effective embedment length of plate or headed bolts, l_b, shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the plate or head of the anchor bolt.~~

~~**2.1.4.2.2.3** The effective embedment length of bent anchors, l_b, shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the bent end minus one anchor bolt diameter.~~

~~**2.1.4.2.3** The allowable load in shear, where l_{be} equals or exceeds 12 bolt diameters, shall be the lesser of that given by Eq. (2-5) or Eq. (2-6).~~

~~$$B_v = 350\sqrt[4]{f'_m}A_b \quad (2\text{-}5)$$~~

~~$$B_v = 0.12A_b f_y \quad (2\text{-}6)$$~~

~~Where l_{be} is less than 12 bolt diameters, the value of B_v in Eq. (2-5) shall be reduced by linear interpolation to zero at an l_{be} distance of 1 in. (25.4 mm).~~

~~**2.1.4.2.4** *Combined shear and tension* Anchors in Section 2.1.4.2 subjected to combined shear and tension shall be designed to satisfy Eq. (2-7).~~

~~$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \le 1 \quad (2\text{-}7)$$~~

2.1.5 *Multiwythe walls*

2.1.5.1 Design of walls composed of more than one wythe shall comply with the provisions of this section.

2.1.5.2 *Composite action*

2.1.5.2.1 Multiwythe walls designed for composite action shall have collar joints either:

(a) crossed by connecting headers, or

(b) filled with mortar or grout and connected by wall ties.

2.1.5.2.2 Shear stresses developed in the planes of interfaces between wythes and collar joints or within headers shall not exceed the following:

(a) mortared collar joints, 5 psi (34.5 kPa).

(b) grouted collar joints, 10 psi (69.0 kPa).

(c) headers, $\sqrt{\text{unit compressive strength of header}}$, psi (MPa) (over net area of header).

2.1.5.2.3 Headers of wythes bonded by headers shall meet the requirements of Section 2.1.5.2.2 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.

2.1.5.2.4 Wythes not bonded by headers shall meet the requirements of Section 2.1.5.2.2 and shall be bonded by wall ties provided as follows:

Wire size	*Minimum number of wall ties required*
W1.7 (MW11)	one per $2^2/_3$ ft^2 (0.25 m^2) of wall
W2.8 (MW18)	one per $4^1/_2$ ft^2 (0.42 m^2) of wall

The maximum spacing between ties shall be 36 in. (914 mm) horizontally and 24 in. (610 mm) vertically.

The use of rectangular wall ties to tie walls made with any type of masonry units is permitted. The use of Z wall ties to tie walls made with other than hollow masonry units is permitted. Cross wires of joint reinforcement are permitted to be used instead of wall ties.

2.1.5.3 *Noncomposite action* — Masonry designed for noncomposite action shall comply with the following provisions:

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

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

(a) Collar joints shall not contain headers, grout, or mortar.

(b) Gravity loads from supported horizontal members shall be resisted by the wythe nearest to the center of span of the supported member. Any resulting bending moment about the weak axis of the wall shall be distributed to each wythe in proportion to its relative stiffness.

(c) Loads acting parallel to the plane of a wall shall be carried only by the wythe on which they are applied. Transfer of stresses from such loads between wythes shall be neglected.

(d) Loads acting transverse to the plane of a wall shall be resisted by all wythes in proportion to their relative flexural stiffnesses.

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

2.1.5.3.2 Wythes of walls designed for noncomposite action shall be connected by wall ties meeting the requirements of Section 2.1.5.2.4 or by adjustable ties. Where the cross wires of joint reinforcement are used as ties, the joint reinforcement shall be ladder-type or tab-type. Wall ties shall be without cavity drips.

Adjustable ties shall meet the following requirements:

(a) One tie shall be provided for each 1.77 ft^2 (0.16 m^2) of wall area.

(b) Horizontal and vertical spacing shall not exceed 16 in. (406 mm).

(c) Adjustable ties shall not be used when the misalignment of bed joints from one wythe to the other exceeds $1^1/_4$ in. (31.8 mm).

(d) Maximum clearance between connecting parts of the tie shall be $^1/_{16}$ in. (1.6 mm).

(e) Pintle ties shall have at least two pintle legs of wire size W2.8 (MW18).

2.1.6 *Columns*

Design of columns shall meet the ~~general~~ requirements of Section 1.14 and the additional requirements of ~~this section~~Section 2.1.6. ~~1 or the requirements of Section 2.1.6.2~~.

~~**2.1.6.1** *General column design*~~

~~**2.1.6.1** Minimum side dimension shall be 8 in. (203 mm) nominal.~~

2.1.6.~~1.~~12 The ratio between the effective height and least nominal dimension shall not exceed 25.

2.1.6.~~1.~~32 Columns shall be designed to resist applied loads. As a minimum, columns shall be designed to resist loads with an eccentricity equal to 0.1 ~~times~~ multiplied by each side dimension. Consider each axis independently.

~~**2.1.6.2** *Lightly loaded columns* — Masonry columns used only to support light frame roofs of carports, porches, sheds or similar structures assigned to Seismic Design Category A, B, or C, which are subject to unfactored gravity loads not exceeding 2000 pounds (8900 N) 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 grouted solid.~~

~~**2.1.6.4** Vertical column reinforcement shall not be less than $0.0025A_g$ nor exceed $0.04A_g$. The minimum number of bars shall be four.~~

~~**2.1.6.5** *Lateral ties* Lateral ties shall conform to the following:~~

~~(a) Longitudinal 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. Lateral ties shall be placed in either a mortar joint or in grout. Where longitudinal bars are located around the perimeter of a circle, a complete circular lateral tie is permitted. Lap length for circular ties shall be 48 tie diameters.~~

~~(d) Lateral ties shall be located vertically not more than one-half lateral tie spacing above the top of footing or slab in any story, and shall be spaced not more than one-half a lateral tie spacing below the lowest horizontal reinforcement in beam, girder, slab, or drop panel above.~~

~~(e) Where beams or brackets frame into a column from four directions, lateral ties shall be permitted to be terminated not more than 3 in. (76.2 mm) below the lowest reinforcement in the shallowest of such beams or brackets.~~

2.1.7 *Pilasters*

2.1.7.1 Walls interfacing with pilasters shall not be considered as flanges, unless the provisions of Section 1.9.4.2 are met.

2.1.7.2 Where vertical reinforcement is provided to resist axial compressive stress, lateral ties shall meet all applicable requirements of Section ~~2.1.6.5~~1.~~16~~14.1.3.

2.1.8 *Load transfer at horizontal connections*

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

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

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

2.1.9 *Concentrated loads*

~~**2.1.9.1** For computing compressive stress f_a for walls laid in running bond, concentrated loads shall not be distributed over the length of supporting wall in excess of the length of wall equal to the width of bearing areas plus four times~~ multiplied by ~~the thickness of the supporting wall, but not to exceed the center-to-center distance between concentrated loads.~~

~~**2.1.9.2** Bearing stresses shall be computed by distributing the bearing load over an area determined as follows:~~

~~(a) The direct bearing area A_1, or~~

~~(b) $A_1\sqrt{A_2 / A_1}$ but not more than $2A_1$, where A_2 is the supporting surface wider than A_1 on all sides, or A_2 is the area of the lower base of the largest frustum of a right pyramid or cone having A_1 as upper base sloping at 45 degrees from the horizontal and wholly contained within the support. For walls in other than running bond, area A_2 shall terminate at head joints.~~

~~**2.1.9.3**~~ Bearing stresses computed over the bearing area, A_{br}, as defined in Section 1.9.5, shall not exceed $0.25 f'_m$.

2.1.10 *Development of reinforcement embedded in grout*

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

2.1.10.2 *Development of wires in tension* — The development length of wire shall be determined by Eq. (2-~~8~~11), but shall not be less than 6 in. (152 mm).

$$l_d = 0.0015\, d_b F_s \qquad (2\text{-}~~8~~11)$$

When epoxy-coated wires are used, development length determined by Eq. (2-~~8~~11) shall be increased by 50 percent.

2.1.10.3 *Development of bars in tension and compression* — The required development length of reinforcing bars shall be determined by Eq. (2-~~9~~12), but shall not be less than 12 in. (305 mm).

$$l_d = \frac{0.13\, d_b^{\,2} f_y \gamma}{K \sqrt{f_m'}} \qquad (2\text{-}\sout{9}12)$$

K shall not exceed the lesser of the masonry cover, clear spacing between adjacent reinforcement, nor 5 ~~times multiplied by~~ d_b.

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

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

γ = 1.5 for No. 8 (M#25) through No. 11 (M#36) bars.

When epoxy-coated reinforcing bars are used, development length determined by Eq. (2-~~9~~12) shall be increased by 50 percent.

2.1.10.4 *Embedment of flexural reinforcement*

2.1.10.4.1 *General*

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

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

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

2.1.10.4.1.4 Continuing reinforcement shall extend a distance l_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure as required by Section 2.1.10.2 or 2.1.10.3.

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

(a) Shear at the cutoff point does not exceed two-thirds of the allowable shear at the section considered.

(b) Stirrup area in excess of that required for shear is provided along each terminated bar or wire over a distance from the termination point equal to three-fourths the effective depth of the

member. Excess stirrup area, A_v, shall not be less than 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 three-fourths the allowable shear at the section considered.

2.1.10.4.1.6 Anchorage complying with Section 2.1.10.2 or 2.1.10.3 shall be provided for tension reinforcement in corbels, deep flexural members, variable-depth arches, members where flexural reinforcement is not parallel with the compression face, and in other cases where the stress in flexural reinforcement does not vary linearly in proportion to the moment.

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

2.1.10.4.3 *Development of negative moment reinforcement*

2.1.10.4.3.1 Negative moment reinforcement in a continuous, restrained, or cantilever member shall be anchored in or through the supporting member in accordance with the provisions of Section 2.1.10.1.

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

2.1.10.5 *Hooks*

2.1.10.5.1 Standard hooks in tension shall be considered to develop an equivalent embedment length, l_e, equal to 11.25 d_b.

2.1.10.5.2 The effect of hooks for bars in compression shall be neglected in design computations.

2.1.10.6 *Development of shear reinforcement*

2.1.10.6.1 *Bar and wire reinforcement*

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

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

(a) A standard hook plus an effective embedment of 0.5 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).

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

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

2.1.10.6.1.5 Pairs of U-stirrups or ties placed to form a closed unit shall be considered properly spliced when length of laps are 1.7 l_d. In grout at least 18 in. (457 mm) deep, such splices with $A_v f_y$ not more than 9,000 lb (40 032 N) per leg shall be permitted to be considered adequate if legs extend the full available depth of grout.

2.1.10.6.2 *Welded wire fabric*

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

(a) Two longitudinal wires at a 2-in. (50.8-mm) spacing along the member at the top of the U, or

(b) One longitudinal wire located not more than $d/4$ from the compression face and a second wire closer to the compression face and spaced not less than 2 in. (50.8 mm) from the first wire. The second wire shall be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend not less than $8d_b$.

2.1.10.6.2.2 For each end of a single leg stirrup of welded smooth or deformed wire fabric, there shall be two longitudinal wires spaced a minimum of 2 in. (50.8 mm) with the inner wire placed at a distance at least $d/4$ or 2 in. (50.8 mm) from middepth of member, $d/2$. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

2.1.10.7 *Splices of reinforcement* — Lap splices, welded splices, or mechanical connections are permitted in accordance with the provisions of this section. All wWelding shall conform to AWS D1.4.

2.1.10.7.1 *Lap splices*

2.1.10.7.1.1 The minimum length of lap for bars in tension or compression shall be determined by Eq. (2-912), but not less than 12 in. (305 mm).

2.1.10.7.1.2 Bars spliced by noncontact lap splices shall not be spaced transversely farther apart than one-fifth the required length of lap nor more than 8 in. (203 mm).

2.1.10.7.2 *Welded splices* — Welded splices shall have the bars butted and welded to develop in tension at least 125 percent of the specified yield strength of the bar.

2.1.10.7.3 *Mechanical connections* — Mechanical connections 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.

2.1.10.7.4 *End-bearing splices*

2.1.10.7.4.1 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.

2.1.10.7.4.2 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.

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

2.2 — Unreinforced masonry

2.2.1 *Scope*

This section provides requirements for unreinforced masonry as defined in Section 1.6, except as otherwise indicated in Section 2.2.4.

2.2.2 *Stresses in reinforcement*

The effect of stresses in reinforcement shall be neglected.

2.2.3 *Axial compression and flexure*

2.2.3.1 Members subjected to axial compression, flexure, or to combined axial compression and flexure shall be designed to satisfy Eq. (2-~~10~~13) and Eq. (2-~~11~~14).

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1 \qquad (2\text{-}\sout{10}13)$$

$$P \le (1/4) P_e \qquad (2\text{-}\sout{11}14)$$

where:

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

$$F_a = (1/4) f'_m \left[1 - \left(\frac{h}{140r} \right)^2 \right] \qquad (2\text{-}\sout{12}15)$$

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

$$F_a = (1/4)\, f'_m \left(\frac{70r}{h} \right)^2 \qquad (2\text{-}\sout{13}16)$$

(c) $F_b = (1/3)\, f'_m$ (2-~~14~~17)

(d) $P_e = \frac{\pi^2 E_m I_n}{h^2} \left(1 - 0.577 \frac{e}{r} \right)^3$ (2-~~15~~18)

2.2.3.2 *Bending* — Allowable tensile stresses for masonry elements subjected to out-of-plane or in-plane bending shall be in accordance with the values in Table 2.2.3.2. For grouted stack bond masonry, tension parallel to the bed joints shall be assumed to be resisted only by the minimum cross-sectional area of continuous grout ~~section~~that is parallel to the bed joints.

2.2.4 *Axial tension*

The tensile strength of unreinforced masonry shall be neglected in design when the masonry is subjected to axial tension forces.

2.2.5 *Shear*

2.2.5.1 Shear stresses due to forces acting in the direction considered shall be computed in accordance with Section 1.9.1 and determined by Eq. (2-1619).

$$f_v = \frac{VQ}{I_n b} \qquad (2\text{-}1619)$$

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

(a) $1.5\sqrt{f'_m}$

(b) 120 psi (827 kPa)

(c) For running bond masonry not grouted solid;

37 psi + 0.45 N_v/A_n

(d) For stack bond masonry with open end units and grouted solid;

37 psi + 0.45 N_v/A_n

(e) For running bond masonry grouted solid;

60 psi + 0.45 N_v/A_n

(f) For stack bond masonry other than open end units grouted solid;

15 psi (103 kPa)

Table 2.2.3.2 — Allowable flexural tensile stresses for clay 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	40 (276)	30 (207)	24 (166)	15 (103)
Hollow units[1]				
Ungrouted	25 (172)	19 (131)	15 (103)	9 (62)
Fully grouted	65 (448)	63 (434)	61 (420)	58 (400)
Parallel to bed joints in running bond				
Solid units	80 (552)	60 (414)	48 (331)	30 (207)
Hollow units				
Ungrouted and partially grouted	50 (345)	38 (262)	30 (207)	19 (131)
Fully grouted	80 (552)	60 (414)	48 (331)	30 (207))
Parallel to bed joints in stack bond				
Continuous grout section parallel to bed joints	100 (690)	100 (690)	100 (690)	100 (690)
Other	0(0)	0(0)	0(0)	0(0)

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

2.3 — Reinforced masonry

2.3.1 *Scope*

This section provides requirements for the design of structures neglecting the contribution of tensile strength of masonry, except as provided in Section 2.3.5.

2.3.2 *Steel reinforcement — Allowable stresses*

2.3.2.1 *Tension* — Tensile stress in reinforcement shall not exceed the following:

(a) Grade 40 or Grade 50 reinforcement

.....20,000 psi (137.9 MPa)

(b) Grade 60 reinforcement 24,000 psi (165.5 MPa)

(c) Wire joint reinforcement 30,000 psi (206.9 MPa)

2.3.2.2 *Compression*

2.3.2.2.1 The compressive resistance of steel reinforcement shall be neglected unless lateral reinforcement is provided in compliance with the requirements of Section ~~2.1.6.5~~1.14.1.3.

2.3.2.2.2 Compressive stress in reinforcement shall not exceed the lesser of 0.4 f_y or 24,000 psi (165.5 MPa).

2.3.3 *Axial compression and flexure*

2.3.3.1 Members subjected to axial compression, flexure, or combined axial compression and flexure shall be designed in compliance with Sections 2.3.3.2 through ~~2.3.3.4~~2.3.3.3.

2.3.3.2 *Allowable forces and stresses*

2.3.3.2.1 The compressive force in reinforced masonry due to axial load only shall not exceed that given by Eq. (2-~~17~~20) or Eq. (2-~~18~~21):

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

$$P_a = (0.25 f'_m A_n + 0.65 A_{st} F_s)\left[1 - \left(\frac{h}{140 r}\right)^2\right] \quad (2\text{-}\sout{17}20)$$

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

$$P_a = (0.25 f'_m A_n + 0.65 A_{st} F_s)\left(\frac{70 r}{h}\right)^2 \quad (2\text{-}\sout{18}21)$$

2.3.3.2.2 The compressive stress in masonry due to flexure or due to flexure in combination with axial load shall not exceed $(^1/_3)\, f'_m$ provided the calculated compressive stress due to the axial load component, f_a, does not exceed the allowable stress, F_a, in Section 2.2.3.1.

~~**2.3.3.3** *Effective compressive width per bar*~~

~~**2.3.3.3.1** In running bond masonry, and masonry in other than running bond with bond beams spaced not more than 48 in. (1219 mm) center-to-center, the width of the compression area used in stress calculations shall not exceed the least of:~~

~~(a) Center-to-center bar spacing.~~

~~(b) Six times the nominal wall thickness.~~

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

~~**2.3.3.3.2** In masonry in other than running bond, with bond beams spaced more than 48 in. (1219 mm) center-to-center, the width of the compression area used in stress calculations shall not exceed the length of the masonry unit.~~

~~**2.3.3.4**~~**2.3.3.3** *Beams* — Design of beams shall meet the requirements of Section 1.13 and the additional requirements of Section 2.3.3.3.

~~**2.3.3.4.1**~~**2.3.3.3.1** ~~Span length of members not built integrally with supports shall be taken as the clear span plus depth of member, but need not exceed the distance between centers of supports.~~

~~**2.3.3.4.2** In analysis of members that are continuous over supports for determination of moments, span length shall be taken as the distance between centers of supports.~~

~~**2.3.3.4.3**~~ Length of bearing of beams on their supports shall be a minimum of 4 in. (102 mm) in the direction of span.

~~**2.3.3.4.4**~~**2.3.3.3.2** ~~The compression face of beams shall be laterally supported at a maximum spacing of 32 times the beam thickness.~~

~~**2.3.3.4.5**~~**2.3.3.3.3** ~~—~~ Beams shall be designed to meet the deflection requirements of Section 1.10.1.

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

$$\rho_{max} = \frac{nf'_m}{2f_y\left(n + \frac{f_y}{f'_m}\right)} \qquad (2\text{-}1922)$$

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

2.3.4 *Axial tension and flexural tension*

Axial tension and flexural tension shall be resisted entirely by steel reinforcement.

2.3.5 *Shear*

2.3.5.1 Members that are not subjected to flexural tension shall be designed in accordance with the requirements of Section 2.2.5 or shall be designed in accordance with the following:

2.3.5.1.1 Reinforcement shall be provided in accordance with the requirements of Section 2.3.5.3.

2.3.5.1.2 The calculated shear stress, f_v, shall not exceed F_v, where F_v is determined in accordance with Section 2.3.5.2.3.

2.3.5.2 Members subjected to flexural tension shall be reinforced to resist the tension and shall be designed in accordance with the following:

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

$$f_v = \frac{V}{bd} \qquad (2\text{-}192023)$$

2.3.5.2.2 Where reinforcement is not provided to resist all of the calculated shear, f_v shall not exceed F_v, where:

(a) for flexural members

$$F_v = \sqrt{f'_m} \qquad (2\text{-}20124)$$

but shall not exceed 50 psi (345 kPa).

(b) for shear walls,

where, $M/Vd < 1$,

$$F_v = (1/3)[4 - (M/Vd)]\sqrt{f'_m} \qquad (2\text{-}21225)$$

but shall not exceed $80 - 45(M/Vd)$ psi

where, $M/Vd \geq 1$,

$$F_v = \sqrt{f'_m} \qquad (2\text{-}\sout{22}326)$$

but shall not exceed 35 psi (241 kPa).

2.3.5.2.3 Where shear reinforcement is provided in accordance with Section 2.3.5.3 to resist ~~all of~~ the entire calculated shear, f_v shall not exceed F_v, where:

(a) for flexural members:

$$F_v = 3.0\sqrt{f'_m} \qquad (2\text{-}\sout{23}427)$$

but shall not exceed 150 psi (1034 kPa).

(b) for shear walls:

where, $M/Vd < 1$,

$$F_v = (\tfrac{1}{2})[4 - (M/Vd)]\sqrt{f'_m} \qquad (2\text{-}\sout{24}528)$$

but shall not exceed $120 - 45(M/Vd)$ psi

where $M/Vd \geq 1$,

$$F_v = 1.5\sqrt{f'_m} \qquad (2\text{-}\sout{25}629)$$

but shall not exceed 75 psi (517 kPa).

2.3.5.2.4 The ratio M/Vd shall always be taken as a positive number.

2.3.5.3 Minimum area of shear reinforcement required by Section 2.3.5.1 or 2.3.5.2.3 shall be determined by the following:

$$A_v = \frac{Vs}{F_s d} \qquad (2\text{-}\sout{26}730)$$

2.3.5.3.1 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).

2.3.5.3.2 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).

2.3.5.4 In composite masonry walls, shear stresses developed in the planes of interfaces between wythes and filled collar joints or between wythes and headers shall meet the requirements of Section 2.1.5.2.2.

2.3.5.5 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 designed for the same shear as that computed 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 member, and

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

CHAPTER 3
STRENGTH DESIGN OF MASONRY

3.1 — General

3.1.1 *Scope*

This Chapter provides minimum requirements for strength design of masonry. Masonry design by the strength design method shall comply with the requirements of Chapter 1, Section 3.1, and either Section 3.2 or 3.3.

3.1.2 *Required strength*

Required strength shall be determined in accordance with the strength design load combinations of the legally adopted building code. When the legally adopted building code does not provide factored load combinations, structures and members shall be designed to resist the combination of loads specified in ASCE 7~~-02~~ for strength design. Members subject to compressive axial load shall be designed for the factored moment accompanying the factored axial load. The factored moment, M_u, shall include the moment induced by relative lateral displacement.

3.1.3 *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 3.1.4.

~~The design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times~~multiplied by the nominal flexural strength, M_n, ~~of the member, except that the nominal shear strength, V_n, need not exceed 2.5 times~~multiplied by required shear strength, V_u.~~

~~**3.1.3.1** *Seismic design provisions* — At each story level, at least 80 percent of the lateral stiffness shall be provided by lateral-force-resisting walls. Along each column line at a particular story level, at least 80 percent of the lateral stiffness shall be provided by lateral-force-resisting walls.~~

~~*Exception*: Where seismic loads are determined based on a seismic response modification factor, R, not greater than 1.5, piers and columns are permitted to be used to provide seismic load resistance.~~

3.1.4 *Strength-reduction factors*

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

3.1.4.2 *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.

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

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

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

3.1.5 *Deformation requirements*

~~**3.1.5.1** *Drift limits* — Under loading combinations that include earthquake, masonry structures shall be designed so the calculated story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from the legally adopted building code. When the legally adopted building code does not provide allowable story drifts, structures shall be designed so the calculated story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from ASCE 7-02.~~

~~For determining drift, the calculated deflection shall be multiplied by C_d as indicated in the legally adopted building code. When the legally adopted building code does not provide C_d values, the deflection amplification factors shall be taken from ASCE 7-02.~~

~~**3.1.5.2** *Deflection of unreinforced (plain) masonry* — Deflection calculations for unreinforced (plain) masonry members shall be based on uncracked section properties.~~

3.1.5.~~3~~2 *Deflection of reinforced masonry* — Deflection calculations for reinforced masonry members shall ~~be based on cracked section properties~~consider the effects of cracking and reinforcement on member stiffness. The flexural and shear stiffness properties assumed for deflection calculations shall not exceed one-half of the gross section properties, unless a cracked-section analysis is performed.

3.1.6 *Anchor bolts embedded in grout*

3.1.6.1 *Design requirements* – Anchor bolts shall be designed using either the provisions of 3.1.6.2 or, for headed and bent-bar anchors, by the provisions of Section 3.1.6.3.

3.1.6.2 *Nominal strengths determined by test*

3.1.6.2.1 Anchors shall be tested in accordance with ASTM E 488, except that a minimum of five tests shall be performed. Loading conditions of the test shall be representative of intended use of the anchor bolt.

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

3.1.6.3 *Nominal strengths determined by calculation for headed and bent-bar anchors* – Nominal strengths of headed and bent-bar anchor bolts embedded in grout shall be determined in accordance with the provisions of Sections 3.1.6.3.1 through 3.1.6.3.3.

3.1.6.3.1 *Nominal tensile strength of headed and bent-bar anchor bolts*

The nominal axial tensile strength of headed anchor bolts shall be computed using the provisions of Sections 3.1.6.3.1.1. The nominal axial tensile strength of bent-bar anchor bolts shall be computed using the provisions of Section 3.1.6.3.1.2.

3.1.6.3.1.1 *Nominal axial tensile strength of headed anchor bolts* — The nominal axial tensile strength, B_{an}, of headed anchor bolts embedded in grout shall be determined by Eq. (3-1) (nominal axial tensile strength governed by masonry breakout) or Eq. (3-2) (nominal axial tensile strength governed by steel yielding). The nominal axial tensile strength, B_{an}, shall be the smaller of the values obtained from Eqs. (3-1) and (3-2).

$$B_{anb} = 4 A_{pt} \sqrt{f'_m} \qquad (3\text{-}1)$$

$$B_{ans} = A_b f_y \quad (3\text{-}2)$$

3.1.6.3.1.2 *Nominal axial tensile strength of bent-bar anchor bolts* – The nominal axial tensile strength, B_{an}, for bent-bar anchor bolts embedded in grout shall be determined by Eq. (3-3) (nominal axial tensile strength governed by masonry breakout), Eq. (3-4) (nominal axial tensile strength governed by anchor bolt pullout), or Eq. (3-5) (nominal axial tensile strength governed by steel yielding). The nominal axial tensile strength, B_{an}, shall be the smallest of the values obtained from Eqs. (3-3), (3-4) and (3-5).

$$B_{anb} = 4 A_{pt} \sqrt{f'_m} \quad (3\text{-}3)$$

$$B_{anp} = 1.5 f'_m e_b d_b + 300\pi (l_b + e_b + d_b) d_b \quad (3\text{-}4)$$

$$B_{ans} = A_b f_y \quad (3\text{-}5)$$

3.1.6.3.2 *Nominal 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 Eq. (3-6) (nominal shear strength governed by masonry breakout), Eq. (3-7) (nominal shear strength governed by masonry crushing), Eq. (3-8) (nominal shear strength governed by anchor bolt pryout) or Eq. (3-9) (nominal shear strength governed by steel yielding). The nominal shear strength B_{vn}, shall be the smallest of the values obtained from Eqs. (3-6), (3-7), (3-8) and (3-9).

$$B_{vnb} = 4 A_{pv} \sqrt{f'_m} \quad (3\text{-}6)$$

$$B_{vnc} = 1050 \sqrt[4]{f'_m} A_b \quad (3\text{-}7)$$

$$B_{vnpry} = 2.0 B_{anb} = 8 A_{pt} \sqrt{f'_m} \quad (3\text{-}8)$$

$$B_{vns} = 0.6 A_b f_y \quad (3\text{-}9)$$

3.1.6.3.3 *Combined axial tension and shear* – Anchor bolts subjected to axial tension in combination with shear shall satisfy Eq. (3-10).

$$\frac{b_{af}}{\phi B_{an}} + \frac{b_{vf}}{\phi B_{vn}} \le 1 \quad (3\text{-}10)$$

~~**3.1.6** *Headed and bent-bar anchor bolts*~~

~~All eEmbedded bolts shall be grouted in place with at least ½ in. (12.7 mm) of grout between the bolt and the masonry, except that ¼ in. (6.4 mm) diameter bolts are permitted to be placed in bed joints that are at least ½ in. (12.7 mm) in thickness.~~

~~**3.1.6.1** *Nominal axial tensile strength of headed anchor bolts* The nominal axial tensile strength, B_{an}, of headed anchor bolts embedded in masonry shall be computed by Eq. (3-1) (strength governed by masonry breakout) and Eq. (3-2) (strength governed by steel). In computing the capacity, the smaller of the design strengths shall be used.~~

~~$$B_{an} = 4 A_{pt} \sqrt{f'_m} \quad (3\text{-}1)$$~~

~~$$B_{an} = A_b f_y \quad (3\text{-}2)$$~~

~~**3.1.6.1.1** *Projected area of masonry for headed anchor bolts* The projected area, A_{pt}, in Eq. (3-1) shall be determined by Eq. (3-3).~~

~~$$A_{pt} = \pi l_b^2 \quad (3\text{-}3)$$~~

~~Where the projected areas, A_{pt}, of adjacent headed anchor bolts overlap, the projected area, A_{pt}, of each bolt shall be reduced by one-half of the overlapping area. The portion of the projected area overlapping an open cell, open head joint, or that is outside the wall shall be deducted from the value of A_{pt} calculated using Eq. (3-3).~~

~~**3.1.6.1.2** *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 bearing surface of the anchor head. The minimum effective embedment length for headed anchor bolts resisting axial forces shall be 4 bolt diameters or 2 in. (50.8 mm), whichever is greater.

3.1.6.2 *Nominal axial tensile strength of bent-bar anchor bolts* — The nominal axial tensile strength, B_{an}, for bent-bar anchor bolts (J- or L-bolts) embedded in masonry shall be computed by Eq. (3-4) (strength governed by masonry breakout), Eq. (3-5) (strength governed by steel), and Eq. (3-6) (strength governed by anchor pullout). In computing the capacity, the smaller of the design strengths shall be used.

$$B_{an} = 4A_{pt}\sqrt{f'_m} \quad (3\text{-}4)$$

$$B_{an} = A_b f_y \quad (3\text{-}5)$$

$$B_{an} = 1.5 f'_m e_b d_b + \left[300\pi\left(l_b + e_b + d_b\right)d_b\right] \quad (3\text{-}6)$$

The second term in Eq. (3-6) shall be included only if the specified quality assurance program includes verification that shanks of J- and L-bolts are free of debris, oil, and grease when installed.

3.1.6.2.1 *Projected area of masonry for bent-bar anchor bolts* — The projected area, A_{pt}, in Eq. (3-4) shall be determined by Eq. (3-7).

$$A_{pt} = \pi l_b^2 \quad (3\text{-}7)$$

Where the projected areas, A_{pt}, of adjacent bent-bar anchor bolts overlap, the projected area, A_{pt}, of each bolt shall be reduced by one-half of the overlapping area. That portion of the projected area overlapping an open cell, open head joint, or that is outside the wall shall be deducted from the value of A_{pt} calculated using Eq. (3-7).

3.1.6.2.2 *Effective embedment length of 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 bearing surface of the bent end, minus one anchor bolt diameter. The minimum effective embedment length for bent-bar anchor bolts resisting axial forces shall be 4 bolt diameters or 2 in. (50.8 mm), whichever is greater.

3.1.6.3 *Nominal shear strength of headed and bent-bar anchor bolts* — The nominal shear

~~strength, B_{vn}, shall be computed by Eq. (3-8) (strength governed by masonry breakout) and Eq. (3-9) (strength governed by steel). In computing the capacity, the smaller of the design strengths shall be used.~~

~~$$B_{vn} = 4A_{pv}\sqrt{f'_m} \quad (3\text{-}8)$$~~

~~$$B_{vn} = 0.6A_b f_y \quad (3\text{-}9)$$~~

~~**3.1.6.3.1** *Projected area of masonry* — The area A_{pv} in Eq. (3-8) shall be determined from Eq. (3-10).~~

~~$$A_{pv} = \frac{\pi l_{be}^2}{2} \quad (3\text{-}10)$$~~

~~**3.1.6.3.2** *Minimum effective embedment length* — The minimum effective embedment length for headed or bent-bar anchor bolts resisting shear forces shall be 4 bolt diameters, or 2 in. (50.8 mm), whichever is greater.~~

~~**3.1.6.4** *Combined axial and shear strength of anchor bolts* — Anchor bolts subjected to combined shear and tension shall be designed to satisfy Eq. (3-11).~~

~~$$\frac{b_{af}}{\phi B_{an}} + \frac{b_{vf}}{\phi B_{vn}} \le 1 \quad (3\text{-}11)$$~~

~~ϕB_{an} and ϕB_{vn} used in Eq. (3-11) shall be the governing design tensile and shear strengths, respectively.~~

3.1.7 *Nominal bearing strength*

The nominal bearing strength of masonry shall be computed as ~~follows:~~0.60 f'_m multiplied by the bearing area, A_{br}, as defined in Section 1.9.5.

~~(a) $C_n = 0.60 f'_m A_1$, or~~

~~(b) $C_n = 0.6 f'_m A_1 \sqrt{A_2 / A_1}$ where:~~

~~$A_1\sqrt{A_2 / A_1} \le 2A_1$ and~~

~~A_1 and A_2 are determined in accordance with Section 2.1.9.2.~~

3.1.8 *Material properties*

3.1.8.1 *Compressive strength*

3.1.8.1.1 *Masonry compressive strength* — The specified compressive strength of masonry, f'_m, shall equal or exceed 1,500 psi (10.34 MPa). 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.

3.1.8.1.2 *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).

3.1.8.2 *Masonry modulus of rupture*

~~**3.1.8.2.1** *Bending* —~~ The modulus of rupture, f_r, for masonry elements subjected to out-of-plane or in-plane bending shall be in accordance with the values in Table ~~3.1.8.2.1~~3.1.8.2. For grouted stack bond masonry, tension parallel to the bed joints shall be assumed to be resisted only by the minimum cross-sectional area of continuous ~~horizontal~~ grout ~~section~~that is parallel to the bed joints. ~~The modulus of rupture of the grout shall be taken as 250 psi (1.72 MPa).~~

3.1.8.3 *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 ~~times~~multiplied by the specified yield strength. The compressive resistance of steel reinforcement shall be neglected unless lateral reinforcement is provided in compliance with the requirements of Section ~~2.1.6.5~~1.14.1.3.

Table 3.1.8.2~~.1~~ — Modulus of rupture, ~~(~~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 in running or stack bond				
Solid units	100 (689)	75 (517)	60 (413)	38 (262)
Hollow units[1]				
Ungrouted	63 (431)	48 (331)	38 (262)	23 (158)
Fully grouted	163 (1124)	158 (1089)	153 (1055)	145 (1000)
Parallel to bed joints in running bond				
Solid units	200 (1379)	150 (1033)	120 (827)	75 (517)
Hollow units				
Ungrouted and partially grouted	125 (862)	95 (655)	75 (517)	48 (331)
Fully grouted	200 (1379)	150 (1033)	120 (827)	75 (517)
Parallel to bed joints in stack bond				
Continuous grout section parallel to bed joints	250 (1734)	250 (1734)	250 (1734)	250 (1734)
Other	0 (0)	0 (0)	0 (0)	0 (0)

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

3.2 —Unreinforced (plain) masonry

3.2.1 *Scope*

The requirements of Section 3.2 are in addition to the requirements of Chapter 1 and Section 3.1 and govern masonry design in which masonry is used to resist tensile forces.

3.2.1.1 *Strength for resisting loads* — Unreinforced (plain) masonry members shall be designed using the strength of masonry units, mortar, and grout in resisting design loads.

3.2.1.2 *Strength contribution from reinforcement* — Stresses in reinforcement shall not be considered effective in resisting design loads.

3.2.1.3 *Design criteria* — Unreinforced (plain) masonry members shall be designed to remain uncracked.

3.2.2 *Flexural and axial strength of unreinforced (plain) masonry members*

3.2.2.1 *Design assumptions* — The following assumptions shall apply when determining the flexural and axial strength of unreinforced (plain) masonry members:

(a) Strength design of members for factored flexure and axial load shall be in accordance with principles of engineering mechanics.

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

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

(d) Flexural compressive stress in combination with axial compressive stress in masonry shall be assumed directly proportional to strain. ~~Nominal compressive strength shall not exceed a stress corresponding to 0.80 f'_m. The nominal axial strength P_n shall not exceed that given in Section 3.2.3.~~

~~(e) The nominal flexural tensile strength of masonry shall be determined from Section 3.1.8.2.~~

3.2.2.2 *Nominal strength*

(a) The nominal compressive strength shall not exceed the value corresponding to a compressive strength of 0.80 f'_m.

(b) The nominal flexural tensile strength of masonry shall be determined from Section 3.1.8.2.

3.2.2.3 *Nominal axial strength* — The nominal axial strength, P_n, shall not be taken as the greater than the following:~~*of unreinforced (plain) masonry members*~~

~~Nominal axial strength, P_n, shall be computed using Eq. (3-12) or Eq. (3-13).~~

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

$$P_n = 0.80\left\{0.80 A_n f'_m\left[1-\left(\frac{h}{140r}\right)^2\right]\right\} \quad (3\text{-}\sout{12}11)$$

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

$$P_n = 0.80\left[0.80 A_n f'_m\left(\frac{70r}{h}\right)^2\right] \quad (3\text{-}\sout{13}12)$$

3.2.2.4 *P-Delta effects*

3.2.2.4.1 Members shall be designed for the factored axial load, P_u, and the moment ~~amplified~~magnified for the effects of member curvature, M_c.

3.2.2.4.2 The magnified moment, M_c, shall be determined either by ~~iteration of deflections~~a second-order analysis, or by a first order analysis and ~~direct calculation using~~ Eqs. (3-~~14~~13) and (3-~~15~~14).

$$M_c = \delta M_u \quad (3\text{-}\sout{14}13)$$

$$\delta = \frac{1}{1-\dfrac{P_u}{A_n f'_m\left(\dfrac{70r}{h}\right)^2}} \quad (3\text{-}\sout{15}14)$$

3.2.2.4.3 It shall be permitted to take $\delta = 1$ for members in which $h/r \le 45$.

3.2.2.4.4 It shall be permitted to take $\delta = 1$ for members in which $45 < h/r \le 60$ provided the nominal strength defined in Section 3.2.2.2 is reduced by 10%.

3.2.3 *Axial tension* — The tensile strength of unreinforced masonry shall be neglected in design when the masonry is subjected to axial tension forces.

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

(a) $3.8 A_n \sqrt{f'_m}$

(b) $300 A_n$

(c) For running bond masonry not solidly grouted;

$$56A_n + 0.45N_u$$

(d) For stack bond masonry with open end units and grouted solid;

$$56A_n + 0.45N_u$$

(e) For running bond masonry grouted solid;

$$90A_n + 0.45N_u$$

(f) For stack bond other than open end units grouted solid;

$$23A_n$$

3.3 — Reinforced masonry

3.3.1 *Scope*

The requirements of this Section are in addition to the requirements of Chapter 1 and Section 3.1 and govern masonry design in which reinforcement is used to resist tensile forces.

3.3.2 *Design assumptions*

The following assumptions apply to the design of reinforced masonry:

(a) There is strain continuity between the reinforcement, grout, and masonry such that ~~all applicable~~ loads are resisted in a composite manner.

(b) The nominal strength of reinforced 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 masonry compression fiber shall be assumed to be 0.0035 for clay masonry and 0.0025 for concrete masonry.

(d) Strain in reinforcement and masonry shall be assumed to be directly proportional to the distance from the neutral axis.

(e) Compression and tension stress in reinforcement shall be taken as E_s ~~times~~multiplied by the steel strain, but not greater than f_y.

(f) The tensile strength of masonry shall be neglected in calculating flexural strength but shall be considered in calculating deflection.

(g) The relationship between masonry compressive stress and masonry strain shall be assumed to be defined by the following:

Masonry stress of 0.80 f'_m shall be assumed uniformly distributed over an equivalent compression ~~zone~~ 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 ~~that~~~~that~~ the neutral axis.

3.3.3 *Reinforcement requirements and details*

3.3.3.1 *Reinforcing bar size limitations* — Reinforcing bars used in masonry shall not be larger than No. 9 (M#29). The nominal bar diameter shall not exceed one-eighth of the nominal member thickness and shall not exceed one-quarter of the least clear dimension of the cell, course, or collar joint in which the bar is placed. The area of reinforcing bars placed in a cell or in a course of hollow unit construction shall not exceed 4 percent of the cell area.

3.3.3.2 *Standard hooks* — The equivalent embedment length to develop standard hooks in tension, l_e, shall be determined by Eq. (3-143-1615):

$$l_e = 13 d_b \qquad (3\text{-}143\text{-}1615)$$

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

The required development length of reinforcement shall be determined by Eq. (3-153-1716), but shall not be less than 12 in. (305 mm).

$$l_d = \frac{0.13\, d_b^{\,2} f_y \gamma}{K \sqrt{f'_m}} \qquad (3\text{-}153\text{-}1716)$$

K shall not exceed the lesser of the masonry cover, clear spacing between adjacent reinforcement, nor 5 ~~times~~multiplied by d_b.

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

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

γ = 1.5 for No. 8 (M#25) through No. 9 (M#29) bars.

When epoxy-coated reinforcing bars are used, development length determined by Eq. (3-153-1716) shall be increased by 50 percent.

3.3.3.3.1 Bars spliced by noncontact lap splices shall not be spaced farther apart than one-fifth the required length of lap nor more than 8 in. (203 mm).

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

3.3.3.3.2.1 Except at wall intersections, the end of a horizontal reinforcing bar needed to satisfy shear strength requirements of Section 3.3.4.1.2 shall be bent around the edge

vertical reinforcing bar with a 180-degree hook. The ends of single leg or U-stirrups shall be anchored by one of the following means:

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

(b) For No. 5 (M #16) bars and smaller, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of $l_d/3$. The $l_d/3$ embedment of a stirrup leg shall be taken as the distance between mid-depth of the member, $d/2$, and the start of the hook (point of tangency).

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

3.3.3.3.2.2 At wall intersections, horizontal reinforcing bars needed to satisfy shear strength requirements of Section 3.3.4.1.2 shall be bent around the edge vertical reinforcing bar with a 90-degree standard hook and shall extend horizontally into the intersecting wall a minimum distance at least equal to the development length.

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

(a) The minimum length of lap for bars shall be 12 in. (305 mm) or the development length determined by Eq. (~~3-15~~3-~~17~~16), whichever is greater.

(b) A welded splice shall have the bars butted and welded to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required.

(c) Mechanical splices shall have the bars connected to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required.

3.3.3.5 *Maximum area of flexural tensile reinforcement*

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

(a) A strain gradient shall be assumed, corresponding to a strain in the extreme tensile reinforcement equal to 1.5 ~~times~~multiplied by the yield strain and a maximum strain in the masonry as given by 3.3.2(c).

(b) The design assumptions of Section 3.3.2 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 as 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.

3.3.3.5.2 For intermediate reinforced masonry shear walls subject to in-plane loads where $M_u/V_u d_v \geq 1$, a strain gradient corresponding to a strain in the extreme tensile reinforcement equal to 3 ~~times~~multiplied by the yield strain and a maximum strain in the masonry as given by 3.3.2(c) shall be used. For intermediate reinforced masonry shear walls subject to out-of-plane loads, the provisions of Section 3.3.3.5.1 shall apply.

3.3.3.5.3 For special reinforced masonry shear walls subject to in-plane loads where $M_u/V_u d_v \geq 1$, a strain gradient corresponding to a strain in the extreme tensile reinforcement equal to 4 ~~times~~multiplied by the yield strain and a maximum strain in the masonry as given by 3.3.2(c) shall be used. For special reinforced masonry shear walls subject to out-of-plane loads, the provisions of Section 3.3.3.5.1 shall apply.

3.3.3.5.4 For masonry members where $M_u/V_u d_v \leq 1$ and when designed using $R \leq 1.5$, there is no upper limit to the maximum flexural tensile reinforcement. For masonry members where $M_u/V_u d_v \leq 1$ and when designed using $R \geq 1.5$, the provisions of Section 3.3.3.5.1 shall apply.

3.3.3.6 *Bundling of reinforcing bars* — Reinforcing bars shall not be bundled.

3.3.4 *Design of beams, piers, and columns*

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

3.3.4.1 *Nominal strength*

3.3.4.1.1 *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 3.3.2 and the provisions of Section 3.3.4.1. Using the slenderness-dependent modification factors of Eq. (~~3-16~~3-~~18~~17) $[1-(h/140r)^2)]$ and Eq. (~~3-17~~3-~~19~~18) $(70r/h)^2$, as appropriate, the nominal axial strength shall be modified for the effects of slenderness. 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 Eq. (~~3-16~~3-~~18~~17) or Eq. (~~3-17~~3-~~19~~18), as appropriate.

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

~~$P_n = 0.80\left[0.80 f'_m (A_n - A_s) + f_y A_s\right]\left[1 - \left(\frac{h}{140r}\right)^2\right]$~~ $P_n = 0.80\left[0.80 f'_m (A_n - A_{st}) + f_y A_{st}\right]\left[1 - \left(\frac{h}{140r}\right)^2\right]$ (~~3-16~~3-~~18~~17)

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

~~$P_n = 0.80\left[0.80 f'_m (A_n - A_s) + f_y A_s\right]\left(\frac{70r}{h}\right)^2$~~ $P_n = 0.80\left[0.80 f'_m (A_n - A_{st}) + f_y A_{st}\right]\left(\frac{70r}{h}\right)^2$ (~~3-17~~3-~~19~~18)

3.3.4.1.2 *Nominal shear strength* — Nominal shear strength, V_n, shall be computed using Eq. (~~3-18~~3-~~20~~19) and either Eq. (~~3-19~~3-~~21~~20) or Eq. (~~3-20~~3-~~22~~21), as appropriate.

$$V_n = V_{nm} + V_{ns} \qquad (3\text{-}18\text{3-}2019)$$

where V_n shall not exceed the following:

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

$$V_n \le 6 A_n \sqrt{f'_m} \qquad (3\text{-}19\text{3-}2120)$$

(b) Where $M_u/V_u\, d_v \ge 1.00$

$$V_n \le 4 A_n \sqrt{f'_m} \qquad (3\text{-}20\text{3-}2221)$$

(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 interpolated.

3.3.4.1.2.1 *Nominal masonry shear strength* — Shear strength provided by the masonry, V_{nm}, shall be computed using Eq. (~~3-21~~3-~~23~~22):

$$V_{nm} = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25 P_u \qquad (3\text{-}21\text{3-}2322)$$

$M_u/(V_u\, d_v)$ need not be taken greater than 1.0.

3.3.4.1.2.2 The value of $M_u/(V_u\, d_v)$ shall be taken as a positive number.

3.3.4.1.2.3 *Nominal shear strength provided by reinforcement* — Nominal shear strength provided by shear reinforcement, V_{ns}, shall be computed as follows:

$$V_{ns} = 0.5\left(\frac{A_v}{s}\right) f_y \, d_v \qquad (3\text{-}223\text{-}2423)$$

3.3.4.2 *Beams* — Design of beams shall meet the requirements of Section 1.13 and the additional requirements of Section 3.3.4.2.

3.3.4.2.1 Members designed primarily to resist flexure shall comply with the requirements of Section 3.3.4.2. The factored axial compressive force on a beam shall not exceed $0.05 A_n f'_m$.

3.3.4.2.2 *Longitudinal reinforcement*

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

3.3.4.2.2.2 The nominal flexural strength of a beam shall not be less than 1.3 ~~times~~multiplied by the nominal cracking moment ~~strength~~ of the beam, M_{cr}. The modulus of rupture, f_r, for this calculation shall be determined in accordance with Section 3.1.8.2.

3.3.4.2.2.3 The requirements of Section 3.3.4.2.2.2 need not be applied if at every section the area of tensile reinforcement provided is at least one-third greater than that required by analysis.

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

(a) Transverse reinforcement shall be a single bar with a 180-degree hook at each end.

(b) Transverse reinforcement shall be hooked around the longitudinal reinforcement.

(c) The minimum area of transverse reinforcement shall be $0.0007 bd_v$.

(d) The first transverse bar shall not be located more than one-fourth of the beam depth, d_v, from the end of the beam.

(e) The maximum spacing shall not exceed one-half the depth of the beam nor 48 in. (1219 mm).

3.3.4.2.4 *Construction* — Beams shall be grouted solid.

3.3.4.2.5 *Dimensional limits* — ~~Dimensions shall be in accordance with the~~

following:

(a) The clear distance between locations of lateral bracing of the compression side of the beam shall not exceed 32 times the least width of the compression area.

(b) The nominal depth of a beam shall not be less than 8 in. (203 mm).

3.3.4.3 *Piers*

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

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

(a) At least, oOne bar shall be provided in the each end cells.

(b) The minimum area of longitudinal reinforcement shall be 0.0007 *bd*.

(c)Longitudinal reinforcement shall be uniformly distributed throughout the depth of the element.

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

(a) The nominal thickness of a pier shall not exceed 16 in. (406 mm).

(b) The distance between lateral supports of a pier shall not exceed 25 timesmultiplied by the nominal thickness of a pier except as provided for in Section 3.3.4.3.3(c).

(c) When the distance between lateral supports of a pier exceeds 25 timesmultiplied by the nominal thickness of the pier, design shall be based on the provisions of Section 3.3.5.

(d) The nominal length of a pier shall not be less than three timesmultiplied by its nominal thickness nor greater than six timesmultiplied by its nominal thickness. The clear height of a pier shall not exceed five timesmultiplied by its nominal length.

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

3.3.4.4 *Columns* — Design of columns shall meet the requirements of Section 1.14 and the additional requirements of Section 3.3.4.4.

3.3.4.4.1 *Longitudinal reinforcement* — Longitudinal reinforcement shall be a minimum of four bars, one in each corner of the column, and shall comply with the following:

(a)Maximum reinforcement area shall be determined in accordance with Section 3.3.3.5, but shall not exceed 0.04 A_n.

(b)Minimum reinforcement area shall be 0.0025 A_n.

(c)Longitudinal reinforcement shall be uniformly distributed throughout the depth of the element.

~~**3.3.4.4.2** *Lateral ties* — Lateral ties shall be provided in accordance with Section 2.1.6.5.~~

~~**3.3.4.4.3**~~**2** *Construction* — Columns shall be solid grouted.

3.3.4.4.4~~3~~**2** *Dimensional limits* — Dimensions shall be in accordance with the following:

(a) ~~The nominal width of a column shall not be less than 8 in. (203 mm).~~

~~(b)~~ The distance between lateral supports of a column shall not exceed 30 ~~times~~multiplied by its nominal width.

(~~e~~b) The nominal depth of a column shall not be less than 8 in. (203 mm) and not be greater than three ~~times~~multiplied by its nominal width.

3.3.5 *Wall design for out-of-plane loads*

3.3.5.1 ~~*General*~~ *Scope* — The requirements of Section 3.3.5 are for the design of walls for out-of-plane loads.

~~**3.3.5.2** *Effective compressive width per bar* — The effective compressive width per bar shall be determined in accordance with Section 2.3.3.3.~~

~~**3.3.5.3**~~**3.3.5.2** *Moment and deflection calculations* — Moment and deflection calculations in Sections ~~3.3.5.4~~3.3.5.3 and ~~3.3.5.5~~3.3.5.4 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.

~~**3.3.5.4**~~**3.3.5.3** *Walls with factored axial stress of $0.20 f'_m$ or less* — The procedures set forth in this Section shall be used when the factored axial load stress at the location of maximum moment satisfies the requirement computed by Eq. (~~3-23~~3-~~25~~24).

$$\left(\frac{P_u}{A_g}\right) \le 0.20 f'_m \qquad (\text{~~3-23~~3-~~25~~24})$$

When the slenderness ratio exceeds 30, the factored axial stress shall not exceed $0.05 f'_m$.

Factored moment and axial force shall be determined at the midheight of the wall and shall be used for design. The factored moment, M_u, at the midheight of the wall shall be computed using Eq. (~~3-24~~3-~~26~~25).

$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u \qquad (\text{~~3-24~~3-~~26~~25})$$

Where:

$$P_u = P_{uw} + P_{uf} \qquad (3\text{-}253\text{-}2726)$$

The deflection due to factored loads (δ_u) shall be obtained using Eq. (3-303-3231) and (3-313-3332) and replacing M_{ser} with M_u and δ_s with δ_u.

The design strength for out-of-plane wall loading shall be in accordance with Eq. (3-263-2827).

$$M_u \le \phi M_n \qquad (3\text{-}263\text{-}2827)$$

The nominal moment shall be calculated using Eqs. (3-273-2928) and (3-283-3029) if the reinforcing steel is placed in the center of the wall.

$$M_n = \left(A_s f_y + P_u\right)\left(d - \frac{a}{2}\right) \qquad (3\text{-}273\text{-}2928)$$

$$a = \frac{\left(P_u + A_s f_y\right)}{0.80 f'_m b} \qquad (3\text{-}283\text{-}3029)$$

The nominal shear strength shall be determined by Section 3.3.4.1.2.

3.3.5.53.3.5.4 *Deflection design* — The horizontal midheight deflection, δ_s, under service lateral and service axial loads (without load factors) shall be limited by the relation:

$$\delta_s \le 0.007\, h \qquad (3\text{-}293\text{-}3130)$$

P-delta effects shall be included in deflection calculation. The midheight deflection shall be computed using either Eq. (3-303-3231) or Eq. (3-313-3332), as applicable.

(a) Where $M_{ser} < M_{cr}$

$$\delta_s = \frac{5M_{ser}h^2}{48E_mI_g} \qquad (3-303\text{-}3231)$$

(b) Where $M_{cr} < M_{ser} < M_n$

$$\delta_s = \frac{5M_{cr}h^2}{48E_mI_g} + \frac{5(M_{ser} - M_{cr})h^2}{48E_mI_{cr}} \quad (3-313\text{-}3332)$$

The cracking moment ~~strength~~ of the wall shall be computed using the modulus of rupture, f_r, taken from Table 3.1.8.2.1.

3.3.6 *Wall design for in-plane loads*

3.3.6.1 *Scope* — The requirements of Section 3.3.6 are for the design of walls to resist in-plane loads.

3.3.6.2 *Reinforcement* —The amount of vertical reinforcement shall not be less than one-half the horizontal reinforcement.

3.3.6.3 *Flexural and axial strength* — The nominal flexural and axial strength shall be determined in accordance with Section 3.3.4.1.1.

3.3.6.4 *Shear strength* — The nominal shear strength shall be computed in accordance with Section 3.3.4.1.2.

3.3.6.5 The maximum reinforcement requirements of Section 3.3.3.5 shall not apply if a shear wall is designed to satisfy the requirements of 3.3.6.6 through 3.3.6.10.

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

1. $P_u \le 0.10\, A_g f'_m$ for geometrically symmetrical wall sections

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

2. $\frac{M_u}{V_u l_w} \le 1.0$

or

3. $V_u \le 3A_n\sqrt{f_m'}$ and $\frac{M_u}{V_u l_w} \le 3.0$

3.3.6.7 The need for special boundary elements at the edges of shear walls shall be evaluated in accordance with Section 3.3.6.8 or 3.3.6.9. The requirements of Sections 3.3.6.10 shall also be satisfied.

3.3.6.8 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 3.3.6.9

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

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

and c is calculated for the P_u given by ASCE 7~~-02~~ Strength Design Load Combination 5 ($1.2D + 1.0E + L + 0.2S$) 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 5 is reducible to 0.5, as per exceptions to Section 2.3.2 of ASCE 7~~-02~~.

(b) Where special boundary elements are required by Section 3.3.6.8 (a), the special boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of l_w or $M_u/4V_u$.

3.3.6.9 Shear walls not designed by Section 3.3.6.8 shall have special boundary elements at boundaries and edges around openings in shear walls where the maximum extreme fiber compressive stress, corresponding to factored forces including earthquake effect, exceeds $0.2f'_m$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15f'_m$. Stresses shall be calculated for the factored forces using a

linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in Section 1.9.4.2.3.9.5.2.3 shall be used.

3.3.6.10 Where special boundary elements are required by Section 3.3.6.8 or 3.3.6.9, requirements (a) through (d) in this section shall be satisfied and tests shall be performed to verify the strain capacity of the element:

(a) The special boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of (c - 0.1l_w) and c/2.

(b) In flanged sections, the special boundary element shall include the effective flange width in compression and shall extend at least 12 in. (305 mm) into the web.

(c) Special boundary element transverse reinforcement at the wall base shall extend into the support a minimum of the development length of the largest longitudinal reinforcement in the boundary element unless the special boundary element terminates on a footing or mat, where special boundary element transverse reinforcement shall extend at least 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.

CHAPTER 4
PRESTRESSED MASONRY

4.1 — General

4.1.1 *Scope*

This chapter provides requirements for design of masonry walls that are prestressed with bonded or unbonded prestressing tendons.

4.1.2 Walls shall be designed for strength requirements and checked for service load requirements.

4.1.3 The wall provisions of Chapter 1 and Section 2.1 shall apply to prestressed masonry walls.

4.1.4 The provisions of Section ~~1.14~~1.16~~1.16~~1.17 shall apply to prestressed masonry walls, except that the cross-sectional area of bonded prestressing tendons shall be considered to contribute to the minimum reinforcement requirements of Sections ~~1.14~~1.16~~1.16~~1.17.5 and ~~1.14~~1.16~~1.16~~1.17.6.

4.1.5 The provisions of Section 4.4.3 shall apply ~~instead of the provisions of Section 2.1.3.4.2~~ for the computation of nominal moment strength.

4.1.6 Masonry shall be laid in running bond unless a bond beam or other technique is used to distribute anchorage forces.

4.2 — Design methods

4.2.1 *General*

Prestressed masonry members shall be designed by elastic analysis using loading and load combinations in accordance with the provisions of Sections 1.7 and 2.1.2, except as noted in Section 4.4.3.

4.2.2 *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} .

4.3 — Permissible stresses in prestressing tendons

4.3.1 *Jacking force*

The stress in prestressing tendons due to the jacking force shall not exceed $0.94f_{py}$, nor $0.80f_{pu}$, nor the maximum value recommended by the manufacturer of the prestressing tendons or anchorages.

4.3.2 *Immediately after transfer*

The stress in the prestressing tendons immediately after transfer of the prestressing force to the masonry shall not exceed $0.82f_{py}$ nor $0.74f_{pu}$.

4.3.3 *Post-tensioned masonry members*

At the time of application of prestress, the stress in prestressing tendons at anchorages and couplers shall not exceed $0.78f_{py}$ nor $0.70f_{pu}$.

4.3.4 *Effective prestress*

The computed effective stress in the prestressing tendons under service 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.

4.4 — Axial compression and flexure

4.4.1 *General*

4.4.1.1 Walls subjected to axial compression, flexure, or to combined axial compression and flexure shall be designed according to the provisions of Section 2.2.3, except as noted in Section 4.4.

4.4.1.2 The allowable compressive stresses due to axial loads, F_a, and flexure, F_b, and the allowable axial force in Eq. (2-~~11~~14) shall be permitted to be increased by 20 percent for the stress condition immediately after transfer of prestress.

4.4.1.3 Masonry shall not be subjected to flexural tensile stress from the combination of prestressing force and dead load.

4.4.2 *Service load requirements*

4.4.2.1 For walls with laterally unrestrained prestressing tendons, the prestressing force, P_{ps}, shall be included in the computation of the axial load, P, in Eq. (2-~~11~~14) and in the computation of the eccentricity of the axial load, e, in Eq. (2-15).

4.4.2.2 For walls with laterally restrained prestressing tendons, the prestressing force, P_{ps}, shall not be considered for the computation of the axial load, P, in Eq. (2-~~11~~14). The prestressing force, P_{ps}, shall be considered for the computation of the eccentricity of the axial resultant load, e, in Eq. (2-~~15~~14).

4.4.3 *Strength requirements*

4.4.3.1 Required strength shall be determined in accordance with the factored load combinations of the legally adopted building code. When the legally adopted building code does not provide factored load combinations, structures and members shall be designed to resist the combination of loads specified in ASCE 7~~-02~~ for strength design. Walls subject to compressive axial load shall be designed for the factored design moment and the accompanying factored axial load. The factored moment, M_u, shall include the moment induced by relative lateral displacement.

4.4.3.2 Values of the response modification coefficient (R) and the deflection amplification factor (C_d), indicated in ASCE 7 Table 12.2-1 for ordinary plain (unreinforced) masonry shear walls shall be used in determining base shear and design story drift.

~~**4.4.3.2**~~ **4.4.3.3** The design moment strength shall be taken as the nominal moment strength, M_n, multiplied by a strength-reduction factor (ϕ) of 0.8.

~~**4.4.3.3**~~**4.4.3.4** For cross sections with uniform width, b, over the depth of the compression zone, the depth of the equivalent compression ~~zone~~stress block, a, shall be determined by the following equation:

$$a = \frac{f_{ps} A_{ps} + f_y A_s + P_u}{0.80\, f'_m\, b} \qquad (4\text{-}1)$$

For other cross sections, Eq. (4-1) shall be modified to consider the variable width of compression zone.

4.4.3.44.4.3.5 For walls with (a) uniform width, b, (b) concentric reinforcement and prestressing tendons, and (c) concentric axial load, the nominal moment strength, M_n, shall be computed by the following equation:

$$M_n = \left(f_{ps} A_{ps} + f_y A_s + P_u\right)\left(d - \frac{a}{2}\right) \qquad (4\text{-}2)$$

4.4.3.4.14.4.3.5.1 The quantity a shall be computed according to Section 4.4.3.34.4.3.4 and f_{ps} shall be computed according to Section 4.4.3.64.4.3.7.

4.4.3.5.24.4.3.4.2 The nominal moment strength for other conditions shall be based on static moment equilibrium principles.

4.4.3.5.34.4.3.4.3 The distance d shall be computed 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 shall not exceed the face-shell thickness plus one-half the tendon diameter plus 0.5 in. (12 mm).

4.4.3.5.44.4.3.4.4 When tendons are not placed in the center of the wall, d shall be computed in each direction for out-of-plane bending.

4.4.3.64.4.3.5 The ratio a/d shall not exceed 0.425.

4.4.3.7 **4.4.3.6** *Computation of f_{ps}* for out-of-plane bending

4.4.3.7.14.4.3.6.1 For walls with bonded prestressing tendons, f_{ps} shall be computed based on strain compatibility or shall be taken equal to f_{py}. Instead of a more accurate determination of f_{ps} for members with unbonded prestressing tendons, the following equation shall be used:

4.4.3.7.24.4.3.6.2 For walls with laterally restrained, unbonded prestressing tendons,

$$f_{ps} = f_{se} + (1{,}000{,}000)\left(\frac{d}{l_p}\right)\sqrt{1 - 1.4\left(\frac{f_{pu} A_{ps}}{bd\ f'_m}\right)} \qquad (4\text{-}3)$$

4.4.3.7.34.4.3.6.3 For walls with laterally unrestrained, unbonded prestressing tendons,

$$f_{ps} = f_{se} + (700{,}000)\left(\frac{d}{l_p}\right)\sqrt{1 - 1.4\left(\frac{f_{pu} A_{ps}}{bd\ f'_m}\right)} \qquad (4\text{-}4)$$

4.4.3.7.24.4.3.6.4 In Eq. (4-3) and (4-4), the value of f_{ps} shall be not less than f_{se}, and not larger than f_{py}.

4.4.3.8 *Computation of f_{ps} for shear walls* — For walls with bonded prestressing tendons, f_{ps} shall be computed based on strain compatibility or shall be taken equal to f_{py}. Instead of a more accurate determination, f_{ps} for members with unbonded prestressing tendons shall be f_{se}.

4.5— Axial tension

Axial tension shall be resisted by reinforcement, prestressing tendons, or both.

4.6 — Shear

4.6.1 For walls without bonded mild reinforcement, nominal shear strength, V_n, shall be computed in accordance with Sections 3.2.4a, 3.2.4b, 3.2.4c, and 3.2.4e. N_u shall include the effective prestress force, $A_{ps} f_{se}$.

4.6.2 For walls with bonded mild reinforcement, nominal shear strength, V_n, shall be computed in accordance with Section 3.3.4.1.2.

4.6.2.1 Nominal masonry shear strength, V_{nm}, shall be computed in accordance with Sections 3.3.4.1.2.1 and 3.3.4.1.2.2. P_u shall include the effective prestress force, $A_{ps} f_{se}$.

4.6.2.2 Nominal shear strength provided by reinforcement, V_{ns}, shall be computed in accordance with Section 3.3.4.1.2.3.

4.7 — Deflection

Computation of member deflection shall include camber, the effects of time-dependent phenomena, and P-delta effects.

4.8 — Prestressing tendon anchorages, couplers, and end blocks

4.8.1 Prestressing tendons in masonry construction shall be anchored by either:

(a) mechanical anchorage devices bearing directly on masonry or placed inside a concrete or fully grouted end block, or

(b) bond in reinforced concrete end blocks or members.

4.8.2 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.

4.8.3 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.

4.8.4 *Bearing stresses*

4.8.4.1 Local bearing stress in masonry in prestressing tendon anchorage zones shall be computed based on the contact surface between masonry and the mechanical anchorage device or the concrete end block.

4.8.4.2 Bearing stresses due to maximum jacking force of the prestressing tendon shall not exceed $0.50 f'_{mi}$.

4.9 — Protection of prestressing tendons and accessories

4.9.1 Prestressing tendons, anchorages, couplers, and end fittings in exterior walls exposed to earth or weather, or walls exposed to a mean relative humidity exceeding 75 percent, shall be corrosion-protected.

4.9.2 Corrosion protection of prestressing tendons shall not rely solely on masonry cover.

4.9.3 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.

4.10 — Development of bonded tendons

Development of bonded prestressing tendons in grouted corrugated ducts, anchored in accordance with Section 4.8.1, does not need to be calculated.

CHAPTER 5
EMPIRICAL DESIGN OF MASONRY

5.1 — General

5.1.1 *Scope*

This chapter provides requirements for empirical design of masonry.

5.1.1.1 The provisions of Chapter 1, excluding Sections 1.2.2(c), 1.7, 1.8, and 1.9, shall apply to empirical design, except as specifically stated here.

5.1.1.2 Article 1.4 of ACI 530.1/ASCE 6/ TMS 602 shall not apply to empirically designed masonry.

5.1.2 *Limitations*

5.1.2.1 *Gravity Loads* — The resultant of gravity loads shall be placed within the center third of the wall thickness and within the central area bounded by lines at one-third of each cross-sectional dimension of foundation piers.

5.1.2.2 *Seismic* — Empirical requirements shall not apply to the design or construction of masonry for buildings, parts of buildings or other structures in Seismic Design Categories D, E, or F as defined in ASCE 7~~-02~~, and shall not apply to the design of the seismic-force-resisting system for structures in Seismic Design Categories B or C.

5.1.2.3 *Wind* — Empirical requirements shall not apply to the design or construction of masonry for buildings, portions of buildings, or other structures in locations with the following basic wind speeds as given in ASCE 7~~-02~~:

5.1.2.3.1 *Lateral force-resisting system* — For masonry elements that are part of the lateral force-resisting system where the basic wind speed exceeds 110 mph (177 km/hr).

5.1.2.3.2 *Interior masonry* — For interior masonry elements that are not part of the lateral force-resisting system in buildings other than enclosed as defined by ASCE 7~~-02~~ in:

- Buildings over 180 ft (55.1 m) in height.
- Buildings over 60 ft (18.4 m) in height where the basic wind speed exceeds 90 mph (145 km/hr).
- Buildings over 35 ft (10.7 m) in height where the basic wind speed exceeds 100 mph (161 km/hr).
- Where the basic wind speed exceeds 110 mph (177 km/hr).

5.1.2.3.3 *Exterior masonry:*

5.1.2.3.3.1 For exterior masonry elements that are not part of the lateral force-resisting system and that are more than 35 ft (10.7 m) above ground:

- Buildings over 180 ft (55.1 m) in height.
- Buildings over 60 ft (18.4 m) in height where the basic wind speed exceeds 90 mph (145 km/hr).
- Buildings over 35 ft (10.7 m) in height where the basic wind speed exceeds 100 mph (161 km/hr).

5.1.2.3.3.2 For exterior masonry elements less than or equal to 35 ft (10.7 m) above ground where the basic wind speed exceeds 110 mph (177 km/hr).

5.1.2.4 *Other horizontal loads* — Empirical requirements shall not apply to structures resisting horizontal loads other than permitted wind or seismic loads or foundation walls as provided in Section 5.6.3.

5.1.2.5 *Glass unit masonry* — The provisions of Chapter 5 shall not apply to glass unit masonry.

5.1.2.6 *AAC masonry* — The provisions of Chapter 5 shall not apply to AAC masonry.

5.2 — Height

Buildings relying on masonry walls as part of their lateral load-resisting system shall not exceed 35 ft (10.67 m) in height.

5.3 — Lateral stability

5.3.1 *Shear walls*

Where the structure depends upon masonry walls for lateral stability, shear walls shall be provided parallel to the direction of the lateral forces resisted.

5.3.1.1 In each direction in which shear walls are required for lateral stability, shear walls shall be positioned in at least two separate planes. The minimum cumulative length of shear walls provided shall be 0.4 ~~times~~multiplied by the long dimension of the building. Cumulative length of shear walls shall not include openings or any element whose length is less than one-half its height.

5.3.1.2 Shear walls shall be spaced so that the length-to-width ratio of each diaphragm transferring lateral forces to the shear walls does not exceed values given in Table 5.3.1.

5.3.2 *Roofs*

The roof construction shall be designed so as not to impart out-of-plane lateral thrust to the walls under roof gravity load.

5.4 — Compressive stress requirements

5.4.1 *Calculations*

Dead and live 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. Compressive stresses in masonry due to vertical dead plus live loads (excluding wind or seismic loads) shall be determined in accordance with the following:

(a) Stresses shall be calculated based on specified dimensions.

(b) Calculated compressive stresses for single wythe walls and for multiwythe composite masonry walls shall be determined by dividing the design load by the gross cross-sectional area of the member. The area of openings, chases, or recesses in walls shall not be included in the gross cross-sectional area of the wall.

5.4.2 *Allowable compressive stresses*

The compressive stresses in masonry shall not exceed the values given in Table 5.4.2. In multiwythe walls, the allowable stresses shall be based on the weakest combination of the units and mortar used in each wythe.

5.5 — Lateral support

5.5.1 *Maximum l/t and h/t~~Intervals~~*

Masonry walls without openings shall be laterally supported in either the horizontal or the vertical direction such that *l/t* or *h/t* does ~~at intervals~~ not exceed~~ing those~~ the values given in Table 5.5.1.

Masonry walls with single or multiple openings shall be laterally supported in either the horizontal or vertical direction such that *l/t* or *h/t* does not exceed the values given in Table 5.5.1 divided by $\sqrt{W_T/W_S}$.

~~W_S = Structural Dimension, as shown in Figure 5.5.1-1.~~

~~W_T = Tributary Dimension, as shown in Figure 5.5.1-1.~~

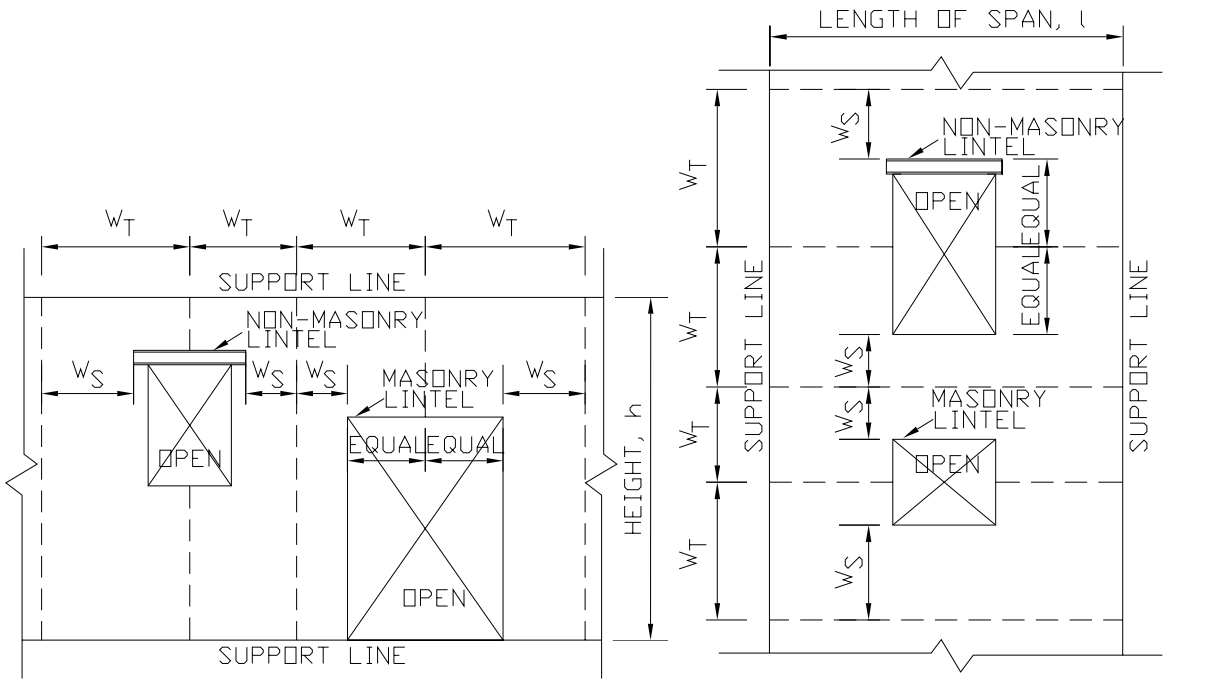
W_S is the dimension of the structural wall strip measured perpendicular to the span of the wall strip and perpendicular to the thickness as shown in Figure 5.5.1-1. W_S is measured from the edge of the opening. W_S shall be no less than $3t$ on each side of each opening. Therefore, at walls with multiple openings, jambs shall be no less than $6t$ between openings. For design purposes, the effective W_S shall not be assumed to be greater than $6t$. At non-masonry lintels, the edge of the opening shall be considered the edge of the non-masonry lintel. W_S shall occur uninterrupted over the full span of the wall.

W_T is the dimension, parallel to W_S, from the center of the opening to the opposite end of W_S as shown in Figure 5.5.1-1. Where there are multiple openings perpendicular to W_S, W_T shall be measured from the center of a virtual opening that encompasses ~~all~~ such openings. ~~All m~~Masonry elements within the virtual opening must be designed in accordance with Chapter 2 or 3.

~~EXCEPTION:~~ For walls with openings that span no more than 4 feet, parallel to W_S, if W_S is no less than 4 feet, then it shall be permitted to ignore the effect of those openings.

The span of openings, parallel to W_S, shall be limited such that the span divided by t does not exceed the values given in Table 5.5.1.

In addition to these limitations, lintels shall be designed for gravity loads in accordance with Section 5.9.2



W_S and W_T for Walls Spanning Vertically W_S and W_T for Walls Spanning Horizontally

Figure 5.5.1-1 — Graphical ~~R~~representation of W_S and W_T

5.5.2 *Cantilever walls*

Except for parapets, the ratio of height-to-nominal-thickness for cantilever walls shall not exceed 6 for solid masonry or 4 for hollow masonry. For parapets see Section 5.6.4.

5.5.3 *Support elements*

Lateral support shall be provided by cross walls, pilasters, buttresses, 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.

5.6 — Thickness of masonry

5.6.1 *General*

Minimum thickness requirements shall be based on nominal dimensions of masonry.

5.6.2 *Minimum thickness*

5.6.2.1 *Bearing Walls* — The minimum thickness of bearing walls of one story buildings shall be 6 in. (152 mm). The minimum thickness of bearing walls of buildings more than one story high shall be 8 in. (203 mm).

5.6.2.2 *Rubble stone walls* — The minimum thickness of rough, random, or coursed rubble stone walls shall be 16 in. (406 mm).

5.6.2.3 *Shear walls* — The minimum thickness of masonry shear walls shall be 8 in. (203 mm).

5.6.2.4 *Foundation walls* — The minimum thickness of foundation walls shall be 8 in. (203 mm).

5.6.2.5 *Foundation piers* — The minimum thickness of foundation piers shall be 8 in. (203 mm).

5.6.2.6 *Parapet walls* — The minimum thickness of parapet walls shall be 8 in. (203 mm).

5.6.2.7 *Change in thickness* — Where walls of masonry of hollow units or masonry bonded hollow walls are decreased in thickness, a course or courses of solid masonry shall be interposed between the wall below and the thinner wall above, or special units or construction shall be used to transmit the loads from face shells or wythes above to those below.

5.6.3 *Foundation walls*

5.6.3.1 Foundation walls shall comply with the requirements of Table 5.6.3.1, which are applicable when:

(a) the foundation wall does not exceed 8 ft (2.44 m) in height between lateral supports,

(b) the terrain surrounding foundation walls is graded to drain surface water away from foundation walls,

(c) backfill is drained to remove ground water away from foundation walls,

(d) lateral support is provided at the top of foundation walls prior to backfilling,

(e) the length of foundation walls between perpendicular masonry walls or pilasters is a maximum of 3 ~~times~~multiplied by the basement wall height,

(f) the backfill is granular and soil conditions in the area are non-expansive, and

(g) masonry is laid in running bond using Type M or S mortar.

Table 5.3.1 — Diaphragm length-to-width ratios

Floor or roof diaphragm construction	Maximum length-to-width ratio of diaphragm panel
Cast-in-place concrete	5:1
Precast concrete	4:1
Metal deck with concrete fill	3:1
Metal deck with no fill	2:1
Wood	2:1

5.6.3.2 Where the requirements of Section 5.6.3.1 are not met, foundation walls shall be designed in accordance with Chapter 1 and Chapter 2, 3, or 4.

5.6.4 *Parapet walls*

The height of parapet walls shall not exceed 3 ~~times~~multiplied by their thickness.

Table 5.4.2 — Allowable compressive stresses for empirical design of masonry

Construction; compressive strength of masonry unit, gross area, psi (MPa)	Allowable compressive stresses[1] based on gross cross-sectional area, psi (MPa)	
	Type M or S mortar	Type N mortar
Solid masonry of brick and other solid units of clay or shale; sand-lime or concrete brick:		
8000 (55.16) or greater	350 (2.41)	300 (2.07)
4500 (31.03)	225 (1.55)	200 (1.38)
2500 (17.23)	160 (1.10)	140 (0.97)
1500 (10.34)	115 (0.79)	100 (0.69)
Grouted masonry of clay or shale; sand-lime or concrete:		
4500 (31.03) or greater	225 (1.55)	200 (1.38)
2500 (17.23)	160 (1.10)	140 (0.97)
1500 (10.34)	115 (0.79)	100 (0.69)
Solid masonry of solid concrete masonry units:		
3000 (20.69) or greater	225 (1.55)	200 (1.38)
2000 (13.79)	160 (1.10)	140 (0.97)
1200 (8.27)	115 (0.79)	100 (0.69)
Masonry of hollow load bearing units of clay or shale[2]:		
2000 (13.79) or greater	140 (0.97)	120 (0.83)
1500 (10.34)	115 (0.79)	100 (0.69)
1000 (6.90)	75 (0.52)	70 (0.48)
700 (4.83)	60 (0.41)	55 (0.38)
Masonry of hollow load bearing concrete masonry units, up to and including 8 in. (203 mm) nominal thickness:		
2000 (13.79) or greater	140 (0.97)	120 (0.83)
1500 (10.34)	115 (0.79)	100 (0.69)
1000 (6.90)	75 (0.52)	70 (0.48)

700 (4.83)	60 (0.41)	55 (0.38)
Masonry of hollow load bearing concrete masonry units, greater than 8 and up to 12 in. (203 to 305 mm) nominal thickness:		
2000 (13.79) or greater	125 (0.86)	110 (0.76)
1500 (10.34)	105 (0.72)	90 (0.62)
1000 (6.90)	65 (0.49)	60 (0.41)
700 (4.83)	55 (0.38	50 (0.35)
Masonry of hollow load bearing concrete masonry units, 12 in. (305 mm) nominal thickness and greater:		
2000 (13.79) or greater	115 (0.79)	100 (0,69)
1500 (10.34)	95 (0.66)	85 (0.59)
1000 (6.90)	60 (0.41)	55 (0.38)
700 (4.83)	50 (0.35)	45 (0.31))
Hollow walls (noncomposite masonry bonded[2 3]):		
Solid units:	160 (1.10)	140 (0.97)
2500 (17.23) or greater	115 (0.79)	100 (0.69)
1500 (10.34)	75 (0.52)	70 (0.48)
Hollow units of clay or shale		
Hollow units of concrete masonry of nominal thickness,	75 (0.52)	70 (0.48)
up to and including 8 in. (203 mm):	70 (0.48)	65 (0.45)
greater than 8 and up to 12 in. (203-305 mm):	60 (0.41)	55(0.38)
12 in. (305 mm) and greater:		
Stone ashlar masonry:		
Granite	720 (4.96)	640 (4.41)
Limestone or marble	450 (3.10)	400 (2.76)
Sandstone or cast stone	360 (2.48)	320 (2.21)
Rubble stone masonry:		
Coursed, rough, or random	120 (0.83)	100 (0.69)

1 Linear interpolation shall be permitted for determining allowable stresses for masonry units having compressive strengths which are intermediate between those given in the table.

2 ~~For hollow concrete masonry units having a nominal thickness of 10 inches (254 mm), the allowable compressive stresses shall be reduced by 10%. For hollow concrete masonry units having a nominal thickness of 12 inches (305 mm) or greater, the allowable compressive stress shall be reduced by 17%.~~

~~3~~ Where floor and roof loads are carried upon one wythe, the gross cross-sectional area is that of the wythe under load; if both wythes are loaded, the gross cross-sectional area is that of the wall minus the area of the cavity between the wythes. Walls bonded with metal ties shall be considered as noncomposite walls, unless collar joints are filled with mortar or grout.

Table 5.5.1 — Wall lateral support requirements

Construction	Maximum *l/t* or *h/t*
Bearing walls	
Solid units or fully grouted	20
~~All other~~Other than solid units or fully grouted	18
Nonbearing walls	
Exterior	18
Interior	36

In computing the ratio for multiwythe walls, use the following thickness:

1. The nominal wall thicknesses for solid walls and for hollow walls bonded with masonry headers (Section 5.7.2).
2. The sum of the nominal thicknesses of the wythes for non-composite walls connected with wall ties (Section 5.7.3).

Table 5.6.3.1 — Foundation wall construction

Wall construction	Nominal wall thickness, in. (mm)	Maximum depth of unbalanced backfill, ft (m)
Hollow unit masonry	8 (203)	5 (1.52)
	10 (254)	6 (1.83)
	12 (305)	7 (2.13)
Solid unit masonry	8 (203)	5 (1.52)
	10 (254)	7 (2.13)
	12 (305)	7 (2.13)
Fully grouted masonry	8 (203)	7 (2.13)
	10 (254)	8 (2.44)
	12 (305)	8 (2.44)

5.7 — Bond

5.7.1 *General*

The facing and backing of multiple wythe masonry walls shall be bonded in accordance with this section.

5.7.2 *Bonding with masonry headers*

5.7.2.1 *Solid units* — Where the facing and backing (adjacent wythes) of solid masonry construction are bonded by means of masonry headers, no less than 4 percent of the wall surface of each face shall be composed of headers extending not less than 3 in. (76.2 mm) into the backing. The distance between adjacent full-length headers shall not exceed 24 in. (610 mm) either vertically or horizontally. In walls in which a single header does not extend through the wall, headers from the opposite sides shall overlap at least 3 in. (76.2 mm), or headers from opposite sides shall be covered with another header course overlapping the header below at least 3 in. (76.2 mm).

5.7.2.2 *Hollow units* — Where two or more hollow units are used to make up the thickness of a wall, the stretcher courses shall be bonded at vertical intervals not exceeding 34 in. (864 mm) by lapping at least 3 in. (76.2 mm) over the unit below, or by lapping at vertical intervals not exceeding 17 in. (432 mm) with units which are at least 50 percent greater in thickness than the units below.

5.7.3 *Bonding with wall ties*

5.7.3.1 When the facing and backing (adjacent wythes) of masonry walls are bonded with wire size W2.8 (MW18) wall ties or metal wire of equivalent stiffness embedded in the horizontal mortar joints, there shall be at least one metal tie for each $4^1/_2$ ft^2 (0.42 m^2) of wall area. The maximum vertical distance between ties shall not exceed 24 in. (610 mm), and the maximum horizontal distance shall not exceed 36 in. (914 mm). Rods or ties bent to rectangular shape shall be used with hollow masonry units laid with the cells vertical. In other walls, the ends of ties shall be bent to 90-degree angles to provide hooks no less than 2 in. (50.8 mm) long. Wall ties shall be without drips. Additional bonding ties shall be provided at openings, spaced not more than 3 ft (0.91 m) apart around the perimeter and within 12 in. (305 mm) of the opening.

5.7.3.2 *Bonding with prefabricated joint reinforcement* — Where the facing and backing (adjacent wythes) of masonry are bonded with prefabricated joint reinforcement, there shall be at least one cross wire serving as a tie for each $2^2/_3$ ft^2 (0.25 m^2) of wall area. The vertical spacing of the joint reinforcement shall not exceed 24 in. (610 mm). Cross wires on prefabricated joint reinforcement shall be not smaller than wire size W1.7 (MW11) and shall be without drips. The longitudinal wires shall be embedded in the mortar.

5.7.4 *Natural or cast stone*

5.7.4.1 *Ashlar masonry* — In ashlar masonry, uniformly distributed bonder units shall be provided to the extent of not less than 10 percent of the wall area. Such bonder units shall extend not less than 4 in. (102 mm) into the backing wall.

5.7.4.2 *Rubble stone masonry* — Rubble stone masonry 24 in. (610 mm) or less in thickness shall have bonder units with a maximum spacing of 3 ft (0.91 m) vertically and 3 ft (0.91 m) horizontally, and if the masonry is of greater thickness than 24 in. (610 mm), shall have one bonder unit for each 6 ft^2 (0.56 m^2) of wall surface on both sides.

5.8 — Anchorage

5.8.1 *General*

Masonry elements shall be anchored in accordance with this section.

5.8.2 *Intersecting walls*

Masonry walls depending upon one another for lateral support shall be anchored or bonded at locations where they meet or intersect by one of the following methods:

5.8.2.1 Fifty percent of the units at the intersection shall be laid in an overlapping masonry bonding pattern, with alternate units having a bearing of not less than 3 in. (76.2 mm) on the unit below.

5.8.2.2 Walls shall be anchored by steel connectors having a minimum section of $^1/_4$ in. (6.4 mm) by $1^1/_2$ in. (38.1 mm) with ends bent up at least 2 in. (50.8 mm), or with cross pins to form anchorage. Such anchors shall be at least 24 in. (610 mm) long and the maximum spacing shall be 4 ft (1.22 m).

5.8.2.3 Walls shall be anchored by joint reinforcement spaced at a maximum distance of 8 in. (203 mm). Longitudinal wires of such reinforcement shall be at least wire size W1.7 (MW11) and shall extend at least 30 in. (762 mm) in each direction at the intersection.

5.8.2.4 Interior nonload bearing walls shall be anchored at their intersection at vertical intervals of not more than 16 in. (406 mm) with joint reinforcement or $^{1}/_{4}$ in. (6.4 mm) mesh galvanized hardware cloth.

5.8.2.5 Other metal ties, joint reinforcement or anchors, if used, shall be spaced to provide equivalent area of anchorage to that required by this section.

5.8.3 *Floor and roof anchorage*

Floor and roof diaphragms providing lateral support to masonry shall be connected to the masonry by one of the following methods:

5.8.3.1 Roof loading shall be determined by the provisions of Section 1.7.2 and, where net uplift occurs, uplift shall be resisted entirely by an anchorage system designed in accordance with the provisions of Sections 2.1 and 2.3, Sections 3.1 and 3.3, or Chapter 4.

5.8.3.2 Wood floor joists bearing on masonry walls shall be anchored to the wall at intervals not to exceed 6 ft (1.83 m) by metal strap anchors. Joists parallel to the wall shall be anchored with metal straps spaced not more than 6 ft (1.83 m) on centers extending over or under and secured to at least 3 joists. Blocking shall be provided between joists at each strap anchor.

5.8.3.3 Steel ~~floor~~ joists that are supported by ~~bearing on~~ masonry walls shall bear on and be connected to steel bearing plates. Maximum joist spacing shall be 6 ft (1.83 m) on center. Each bearing plate shall be anchored to the wall with a minimum of two ½ in. (12.7 mm) diameter bolts ~~$^{3}/_{8}$ in. (9.5 mm) round bars~~, or their equivalent~~, spaced not more than 6 ft (1.83 m) on center~~. Where steel joists are parallel to the wall, anchors shall be located ~~at~~where joist bridging terminates at the wall and additional anchorage shall be provided to comply with Section 5.8.3.4.

5.8.3.4 Roof and floor diaphragms shall be anchored to masonry walls with a minimum of ½ in.~~ $^{1}/_{2}$ in.~~ (12.7-mm) diameter bolts at a maximum spacing of 6 ft (1.83 m) on center or their equivalent. ~~Bolts shall extend and be embedded at least 15 in. (381 mm) into the masonry, or be hooked or welded to not less than 0.20 in.^{2} (129 mm^{2}) of bond beam reinforcement placed not less than 6 in. (152 mm) from the top of the wall.~~

5.8.3.5 Bolts and anchors required by Sections 5.8.3.3 and 5.8.3.4 shall comply with the following:

(a) Bolts and anchors at steel floor joists and floor diaphragms shall be embedded ~~into~~ in the masonry at least 6 in. (152 mm) or shall comply with Section 5.8.3.5 (c).

(b) Bolts at steel roof joists and roof diaphragms shall be embedded ~~into~~in the masonry at least 15 in. (381 mm) or shall comply with Section 5.8.3.5(c).

(c) In lieu of the embedment lengths listed in Sections 5.8.3.5(a) and 5.8.3.5(b), bolts shall be permitted to be hooked or welded to not less than 0.20 $in.^2$ (129 mm^2) of bond beam reinforcement placed not less than 6 in. (152 mm) below joist bearing or bottom of diaphragm.

5.8.4 *Walls adjoining structural framing*

Where walls are dependent upon the structural frame for lateral support, they shall be anchored to the structural members with metal anchors or otherwise keyed to the structural members. Metal anchors shall consist of $^1/_2$-in. (12.7-mm) bolts spaced at 4 ft (1.22 m) on center embedded 4 in. (102 mm) into the masonry, or their equivalent area.

5.9 — Miscellaneous requirements

5.9.1 *Chases and recesses*

Masonry directly above chases or recesses wider than 12 in. (305 mm) shall be supported on lintels.

5.9.2 *Lintels*

The design of masonry lintels shall be in accordance with the provisions of Section ~~2.3.3.4~~2.3.3.3 or Section 3.3.4.2.

5.9.3 *Support on wood*

No masonry shall be supported on wood girders or other forms of wood construction.

CHAPTER 6
VENEER

6.1 — General

6.1.1 *Scope*

This chapter provides requirements for design and detailing of anchored masonry veneer and adhered masonry veneer.

6.1.1.1 The provisions of Chapter 1, excluding Sections 1.2.2(c), 1.7, and 1.9, shall apply to design of anchored and adhered veneer except as specifically stated here.

6.1.1.2 Section 1.11 shall not apply to adhered veneer.

6.1.1.3 Articles 1.4 A and B and 3.4 C of ACI 530.1/ASCE 6/TMS 602 shall not apply to any veneer. Articles 3.4 B and ~~E~~ F shall not apply to anchored veneer. Articles 3.3 B and 3.4 A, B, ~~D~~ E and ~~E~~ F shall not apply to adhered veneer.

6.1.2 *Design of anchored veneer*

Anchored veneer shall meet the requirements of Section 6.1.5 and shall be designed rationally by Section 6.2.1 or detailed by the prescriptive requirements of Section 6.2.2.

6.1.3 *Design of adhered veneer*

Adhered veneer shall meet the requirements of Section 6.1.5, and shall be designed rationally by Section 6.3.1 or detailed by the prescriptive requirements of Section 6.3.2.

6.1.4 *Dimension stone*

Dimension stone veneer is not covered under this Code. Any such system shall be considered a Special System and submitted accordingly to the Building Official.

6.1.5 *General design requirements*

6.1.5.1 Design and detail the backing system of exterior veneer to resist water penetration. Exterior sheathing shall be covered with a water-resistant membrane, unless the sheathing is water resistant and the joints are sealed.

6.1.5.2 Design and detail flashing and weep holes in exterior veneer wall systems to resist water penetration into the building interior. Weepholes shall be at least $^{3}/_{16}$ in. (4.8 mm) in diameter and spaced less than 33 in. (838 mm) on center.

6.1.5.3 Design and detail the veneer to accommodate differential movement.

6.2 — Anchored veneer

6.2.1 *Alternative design of anchored masonry veneer*

The alternative design of anchored veneer, which is permitted under Section 1.3, shall satisfy the following conditions:

(a) Loads shall be distributed through the veneer to the anchors and the backing using principles of mechanics.

(b) Out-of-plane deflection of the backing shall be limited to maintain veneer stability.

(c) ~~All m~~Masonry, other than veneer, shall meet the provisions of Section 1.1.3, excluding subparagraphs (e) and (f).

(d) The veneer is not subject to the flexural tensile stress provisions of Section 2.2 or the nominal flexural tensile strength provisions of Section 3.2.2.

(e) The provisions of Chapter 1, excluding Section 1.2.2(c), Section 6.1, excluding Section 6.1.1.1, Section 6.2.2.9, and Section 6.2.2.10 shall apply.

6.2.2 *Prescriptive requirements for anchored masonry veneer*

6.2.2.1 Except as provided in Section 6.2.2.11, prescriptive requirements for anchored masonry veneer shall not be used in areas where the basic wind speed exceeds 110 mph (177 km/hr) as given in ASCE 7~~-02~~.

6.2.2.2 Connect anchored veneer to the backing with anchors that comply with Section 6.2.2.5 and Article 2.4 of ACI 530.1/ASCE 6/TMS 602.

6.2.2.3 *Vertical support of anchored masonry veneer*

6.2.2.3.1 The weight of anchored veneer shall be supported vertically on concrete or masonry foundations or other noncombustible structural supports, except as permitted in Sections 6.2.2.3.1.1, 6.2.2.3.1.4, and 6.2.2.3.1.5.

6.2.2.3.1.1 Anchored veneer is permitted to be supported vertically by preservative-treated wood foundations. The height of veneer supported by wood foundations shall not exceed 18 ft (5.49 m) above the support.

6.2.2.3.1.2 Anchored veneer with a backing of wood framing shall not exceed the height above the noncombustible foundation given in Table 6.2.2.3.1.

6.2.2.3.1.3 If anchored veneer with a backing of cold-formed steel framing exceeds the height above the noncombustible foundation given in Table 6.2.2.3.1, the weight of the veneer shall be supported by noncombustible construction for each story above the height limit given in Table 6.2.2.3.1.

Table 6.2.2.3.1 — Height limit from foundation

Height at plate, ft (m)	Height at gable, ft (m)
30 (9.14)	38 (11.58)

6.2.2.3.1.4 When anchored veneer is used as an interior finish on wood framing, it shall have a weight of 40 lb/ft^2 (1915 Pa) or less and be installed in conformance with the provisions of this Chapter.

6.2.2.3.1.5 Exterior masonry veneer having an installed weight of 40 psf (195 kg/m^2) or less and height of no more than 12 ft (3.7 m) shall be permitted to be supported on wood construction. A vertical movement joint in the masonry veneer shall be used to isolate the veneer supported by wood construction from that supported by the foundation. Masonry shall be designed and constructed so that masonry is not in direct contact with wood. The horizontally spanning element supporting the masonry veneer shall be designed so that deflection due to dead plus live loads does not exceed *l*/600 or 0.3 in. (7.6 mm).

6.2.2.3.2 When anchored veneer is supported by floor construction, the floor shall be designed to limit deflection as required in Section 1.10.1.

6.2.2.3.3 Provide noncombustible lintels or supports attached to noncombustible framing over openings where the anchored veneer is not self-supporting. The deflection of such lintels or supports shall conform to the requirements of Section 1.10.1.

6.2.2.4 *Masonry units* - Masonry units shall be at least $2^5/_8$ in. (66.7 mm) in actual thickness.

6.2.2.5 *Anchor requirements*

6.2.2.5.1 *Corrugated sheet-metal anchors*

6.2.2.5.1.1 Corrugated sheet-metal anchors shall be at least $^{7}/_{8}$ in. (22 mm) wide, have a base metal thickness of at least 0.03 in. (0.8 mm), and shall have corrugations with a wavelength of 0.3 to 0.5 in. (7.6 to 12.7 mm) and an amplitude of 0.06 to 0.10 in. (1.5 to 2.5 mm).

6.2.2.5.1.2 Corrugated sheet-metal anchors shall be placed as follows:

(a) With solid units, embed anchors in the mortar joint and extend into the veneer a minimum of $1^{1}/_{2}$ 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 $1^{1}/_{2}$ in. (38.1 mm), with at least $^{5}/_{8}$-in. (15.9-mm) mortar or grout cover to the outside face.

6.2.2.5.2 *Sheet-metal anchors*

6.2.2.5.2.1 Sheet-metal anchors shall be at least $^{7}/_{8}$ in. (22.2 mm) wide, shall have a base metal thickness of at least 0.06 in. (1.5 mm), and shall:

(a) have corrugations as given in Section 6.2.2.5.1.1, or

(b) be bent, notched, or punched to provide equivalent performance in pull-out or push-through.

6.2.2.5.2.2 Sheet-metal anchors shall be placed as follows:

(a) With solid units, embed anchors in the mortar joint and extend into the veneer a minimum of $1^{1}/_{2}$ 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 $1^{1}/_{2}$ in. (38.1 mm), with at least $^{5}/_{8}$-in. (15.9-mm) mortar or grout cover to the outside face.

6.2.2.5.3 *Wire anchors*

6.2.2.5.3.1 Wire anchors shall be at least wire size W1.7 (MW11) and have ends bent to form an extension from the bend at least 2 in. (50.8 mm) long.

6.2.2.5.3.2 Wire anchors shall be placed as follows:

(a) With solid units, embed anchors in the mortar joint and extend into the veneer a minimum of $1^{1}/_{2}$ 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 $1^{1}/_{2}$ in. (38.1 mm), with at least $^{5}/_{8}$-in. (15.9-mm) mortar or grout cover to the outside face.

6.2.2.5.4 *Joint reinforcement*

6.2.2.5.4.1 Ladder-type or tab-type joint reinforcement is permitted. Cross wires used to anchor masonry veneer shall be at least wire size W1.7 (MW11) and shall be spaced at a maximum of 16 in. (406 mm) on center. Cross wires shall be welded to longitudinal wires, which shall be at least wire size W1.7 (MW11).

6.2.2.5.4.2 Embed longitudinal wires of joint reinforcement in the mortar joint with at least $^{5}/_{8}$-in. (15.9-mm) mortar cover on each side.

6.2.2.5.5 *Adjustable anchors*

6.2.2.5.5.1 Sheet-metal and wire components of adjustable anchors shall conform to the requirements of Section 6.2.2.5.1, 6.2.2.5.2, or 6.2.2.5.3. Adjustable anchors with joint reinforcement shall also meet the requirements of Section 6.2.2.5.4.

6.2.2.5.5.2 Maximum clearance between connecting parts of the tie shall be $^{1}/_{16}$ in. (1.6 mm).

6.2.2.5.5.3 Adjustable anchors shall be detailed to prevent disengagement.

6.2.2.5.5.4 Pintle anchors shall have at least two pintle legs of wire size W2.8 (MW18) each and shall have an offset not exceeding $1^{1}/_{4}$ in. (31.8 mm).

6.2.2.5.5.5 Adjustable anchors of equivalent strength and stiffness to those specified in Sections 6.2.2.5.5.1 through 6.2.2.5.5.4 are permitted.

6.2.2.5.6 *Anchor spacing*

6.2.2.5.6.1 For adjustable two-piece anchors, anchors of wire size W1.7 (MW11), and 22 gage (0.8 mm) corrugated sheet-metal anchors, provide at least one anchor for each 2.67 ft^2 (0.25 m^2) of wall area.

6.2.2.5.6.2 For other anchors, provide at least one anchor for each 3.5 ft^2 (0.33 m^2) of wall area.

6.2.2.5.6.3 Space anchors at a maximum of 32 in. (813 mm) horizontally and ~~18~~ 25 in. (~~457~~ 635 mm) vertically, but not to exceed the applicable requirements of Section 6.2.2.5.6.1 or 6.2.2.5.6.2.

6.2.2.5.6.4 Provide additional anchors around openings larger than 16 in. (406 mm) in either dimension. Space anchors around perimeter of opening at a maximum of 3 ft (0.91 m) on center. Place anchors within 12 in. (305 mm) of openings.

6.2.2.5.7 *Joint thickness for anchors* — Mortar bed joint thickness shall be at least twice the thickness of the embedded anchor.

6.2.2.6 *Masonry veneer anchored to wood backing*

6.2.2.6.1 Veneer shall be attached with any anchor permitted in Section 6.2.2.5.

6.2.2.6.2 Attach each anchor to wood studs or wood framing with a corrosion-resistant 8d common nail, or with a fastener having equivalent or greater pullout strength. For corrugated sheet-metal anchors, locate the nail or fastener within $^{1}/_{2}$ in. (12.7 mm) of the 90-degree bend in the anchor.

6.2.2.6.3 ~~Maintain~~ When corrugated sheet metal anchors are used, a maximum distance between the inside face of the veneer and outside face of the solid sheathing of 1 in. (25.4 mm) ~~when corrugated sheet metal anchors are used~~shall be specified. ~~Maintain~~ When other anchors are used, a maximum distance between the inside face of the veneer and the wood stud or wood framing of 4½ ~~+/2~~ in. (114 mm) ~~when other anchors are used~~shall be specified. ~~Maintain a~~A 1-in. (25.4-mm) minimum air space shall be specified.

6.2.2.7 *Masonry veneer anchored to steel backing*

6.2.2.7.1 Attach veneer with adjustable anchors.

6.2.2.7.2 Attach each anchor to steel framing with corrosion-resistant screws that have a minimum nominal shank diameter of 0.190 in. (4.8 mm).

6.2.2.7.3 Cold-formed steel framing shall be corrosion resistant and have a minimum base metal thickness of 0.043 in. (1.1 mm).

6.2.2.7.4 ~~Maintain a~~A 4½ ~~+/2~~ in. (114-mm) maximum distance between the inside face of the veneer and the steel framing shall be specified. ~~Maintain a~~A 1-in. (25.4-mm) minimum air space shall be specified.

6.2.2.8 *Masonry veneer anchored to masonry or concrete backing*

6.2.2.8.1 Attach veneer to masonry backing with wire anchors, adjustable anchors, or joint reinforcement. Attach veneer to concrete backing with adjustable anchors.

6.2.2.8.2 ~~Maintain a~~A 4½ ~~+/2~~ in. (114-mm) maximum distance between the inside face of the veneer and the outside face of the masonry or concrete backing shall be specified. ~~Maintain a~~A 1-in. (25.4-mm) minimum air space shall be specified.

6.2.2.9 *Veneer laid in other than running bond* — Anchored veneer laid in other than running bond shall have joint reinforcement of at least one wire, of size W1.7 (MW11), spaced at a maximum of 18 in. (457 mm) on center vertically.

6.2.2.10 *Requirements in seismic areas*

6.2.2.10.1 *Seismic Design Category C*

6.2.2.10.1.1 The requirements of this section apply to anchored veneer for buildings in Seismic Design Category C.

6.2.2.10.1.2 Isolate the sides and top of anchored veneer from the structure so that vertical and lateral seismic forces resisted by the structure are not imparted to the veneer.

6.2.2.10.2 *Seismic Design Category D*

6.2.2.10.2.1 The requirements for Seismic Design Category C and the requirements of this section apply to anchored veneer for buildings in Seismic Design Category D.

6.2.2.10.2.2 Reduce the maximum wall area supported by each anchor to 75 percent of that required in Sections 6.2.2.5.6.1 and 6.2.2.5.6.2. Maximum horizontal and vertical spacings are unchanged.

6.2.2.10.3 *Seismic Design Categories E and F*

6.2.2.10.3.1 The requirements for Seismic Design Category D and the requirements of this section apply to anchored veneer for buildings in Seismic Design Categories E and F.

6.2.2.10.3.2 Support the weight of anchored veneer for each story independent of other stories.

6.2.2.10.3.3 Provide continuous single wire joint reinforcement of wire size W1.7 (MW11) at a maximum spacing of 18 in. (457 mm) on center vertically. Mechanically attach anchors to the joint reinforcement with clips or hooks.

6.2.2.11 *Requirements in areas of high winds* — The following requirements apply in areas where the basic wind speed exceeds 110 mph (177 km/hr) but does not exceed 130 mph (209 km/hr) and the building's mean roof height is less than or equal to 60 ft (18.3 m):

(a) Reduce the maximum wall area supported by each anchor to 70 percent of that required in Sections 6.2.2.5.6.1 and 6.2.2.5.6.2.

(b) Space anchors at a maximum 18 in. (457 mm) horizontally and vertically.

(c) Provide additional anchors around openings larger than 16 in. (406 mm) in either direction. Space anchors around perimeter of opening at a maximum of 24 in. (610 mm) on center. Place anchors within 12 in. (305 mm) of openings.

6.3 — Adhered veneer

6.3.1 *Alternative design of adhered masonry veneer*

The alternative design of adhered veneer, which is permitted under Section 1.3, shall satisfy the following conditions:

(a) Loads shall be distributed through the veneer to the backing using principles of mechanics.

(b) Out-of-plane curvature shall be limited to prevent veneer unit separation from the backing.

(c) ~~All m~~Masonry, other than veneer, shall meet the provisions of Section 1.1.3, excluding subparagraphs (e) and (f).

(d) The veneer is not subject to the flexural tensile stress provisions of Section 2.2 or the nominal flexural tensile strength provisions of Section 3.2.2.

(e) The provisions of Chapter 1, excluding Section 1.2.2(c), and Section 6.1, excluding Section 6.1.1, shall apply.

6.3.2 *Prescriptive requirements for adhered masonry veneer*

6.3.2.1 *Unit sizes* — Adhered veneer units shall not exceed $2^5/_8$ in. (66.7 mm) in specified thickness, 36 in. (914 mm) in any face dimension, nor more than 5 ft^2 (0.46 m^2) in total face area, and shall not weigh more than 15 lb/ft^2 (718 Pa).

6.3.2.2 *Wall area limitations* — The height, length, and area of adhered veneer shall not be limited except as required to control restrained differential movement stresses between veneer and backing.

6.3.2.3 *Backing* — Backing shall provide a continuous, moisture-resistant surface to receive the adhered veneer. Backing is permitted to be masonry, concrete, or metal lath and portland cement plaster applied to masonry, concrete, steel framing, or wood framing.

6.3.2.4 Adhesion developed between adhered veneer units and backing shall have a shear strength of at least 50 psi (345 kPa) based on gross unit surface area when tested in accordance with ASTM C 482, or shall be adhered in compliance with Article 3.3 C of ACI 530.1/ASCE 6/TMS 602.

CHAPTER 7
GLASS UNIT MASONRY

7.1 — General

7.1.1 *Scope*

This chapter provides requirements for empirical design of glass unit masonry as nonload-bearing elements in exterior or interior walls.

7.1.1.1 The provisions of Chapter 1, excluding Sections 1.2.2(c), 1.7, 1.8, and 1.9, shall apply to design of glass unit masonry, except as stated here.

7.1.1.2 Article 1.4 of ACI 530.1/ASCE 6/TMS 602 shall not apply to glass unit masonry.

7.1.2 *General design requirements*

Design and detail glass unit masonry to accommodate differential movement.

7.1.3 *Units*

7.1.3.1 Hollow or solid glass block units shall be standard or thin units.

7.1.3.2 The specified thickness of standard units shall be at least $3^{7}/_{8}$ in. (98.4 mm).

7.1.3.3 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.

7.2 — Panel size

7.2.1 *Exterior standard-unit panels*

The maximum area of each individual standard-unit panel shall be based on the design wind pressure, in accordance with ~~Fig.~~Figure 7.2-1. The maximum dimension between structural supports shall be 25 ft (7.62 m) horizontally or 20 ft (6.10 m) vertically.

7.2.2 *Exterior thin-unit panels*

The maximum area of each individual thin-unit panel shall be 85 ft^2 (7.90 m^2). The maximum dimension between structural supports shall be 15 ft (4.57 m) wide or 10 ft (3.05 m) high. Thin units shall not be used in applications where the design wind pressure exceeds 20 lb/ft^2 (958 Pa).

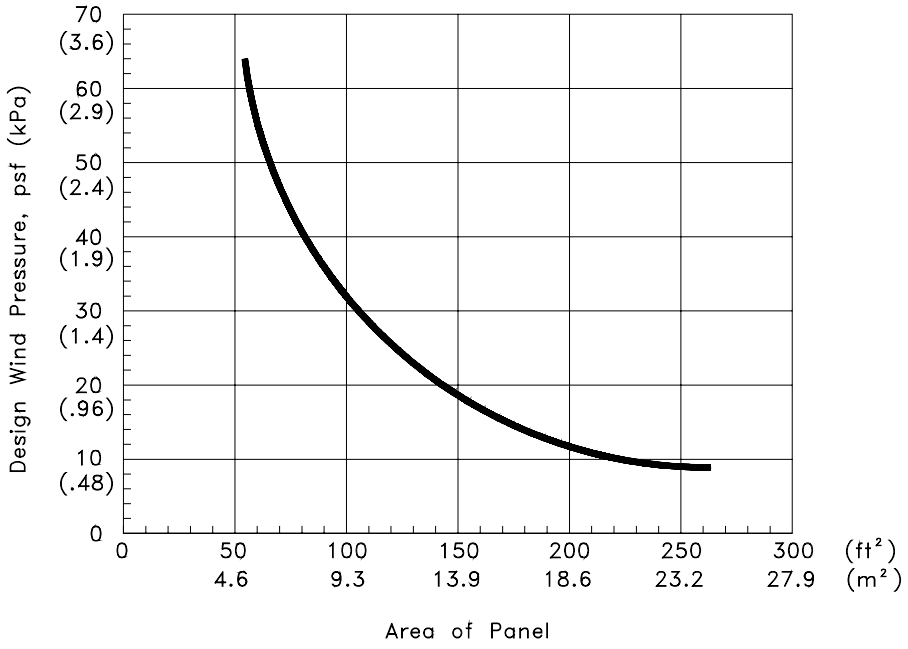
7.2.3 *Interior panels*

7.2.3.1 When the~~The~~ wind pressure does not exceed 10 psf (480 Pa), the maximum area of each individual standard-unit panel shall be 250 ft^2 (23.22 m^2~~).~~) and the~~The~~ maximum area of each thin-unit panel shall be 150 ft^2 (13.94 m^2). The maximum dimension between structural supports shall be 25 ft (7.62 m) wide or 20 ft (6.10 m) high.

~~b.~~**7.2.3.2** When the wind pressure exceeds 10 psf (480 Pa), standard-unit panels shall be designed in accordance with Section 7.2.1 and thin-unit panels shall be designed in accordance with Section 7.2.2.

7.2.4 *Curved panels*

The width of curved panels shall conform to the requirements of Sections 7.2.1, 7.2.2, and 7.2.3, except additional structural supports shall be provided at locations where a curved section joins a straight section and at inflection points in multi-curved walls.



~~Fig.~~*Figure 7.2-1 — Glass ~~U~~unit ~~M~~masonry ~~D~~design ~~W~~wind ~~L~~load ~~R~~resistance*

7.3 — Support

7.3.1 *General requirements*

Glass unit masonry panels shall be isolated so that in-plane loads are not imparted to the panel.

7.3.2 *Vertical*

7.3.2.1 Maximum total deflection of structural members supporting glass unit masonry shall not exceed *l*/600.

7.3.2.2 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.

7.3.2.3 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.

7.3.3 *Lateral*

7.3.3.1 Glass unit masonry panels, more than one unit wide or one unit high, shall be laterally supported along the top and sides of the panel. Lateral support shall be provided by panel anchors along the top and sides spaced not more than 16 in. (406 mm) on center or by channel-type restraints. Glass unit masonry panels shall be recessed at least 1 in. (25.4 mm) within channels and chases. Channel-type restraints must be oversized to accommodate expansion material in the opening, and packing and sealant between the framing restraints and the glass unit masonry perimeter units. Lateral supports for glass unit masonry panels shall be designed to resist applied loads, or a minimum of 200 lb per lineal ft (2919 N/m) of panel, whichever is greater.

7.3.3.2 Glass unit masonry panels that are no more than one unit wide shall conform to the requirements of Section 7.3.3.1, except that lateral support at the top of the panel is not required.

7.3.3.3 Glass unit masonry panels that are no more than one unit high shall conform to the requirements of Section 7.3.3.1, except that lateral support at the sides of the panels is not required.

7.3.3.4 Glass unit masonry panels that are a single glass masonry unit shall conform to the requirements of Section 7.3.3.1, except that lateral support shall not be provided by panel anchors.

7.4 — 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.

7.5 — 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.

7.6 — Mortar

Glass unit masonry shall be laid with Type S or N mortar.

7.7 — 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. Longitudinal wires shall be lapped a minimum of 6 in. (152 mm) at splices. Joint reinforcement shall be placed in the bed joint immediately below and above openings in the panel. The reinforcement shall have not less than two parallel longitudinal wires of size W1.7 (MW11) and have welded cross wires of size W1.7 (MW11).

APPENDIX A

STRENGTH DESIGN OF AUTOCLAVED AERATED CONCRETE (AAC) MASONRY

A.1 — General

A.1.1 *Scope*

This Appendix provides minimum requirements for design of AAC masonry. AAC masonry shall comply with the requirements of Chapter 1, Section A.1, and either Section A.2 or A.3.

A.1.2 *Required strength*

Required strength shall be determined in accordance with the strength design load combinations 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 7-02. Members subject to compressive axial load shall be designed for the maximum design moment accompanying the axial load. The factored moment, M_u, shall include the moment induced by relative lateral displacement.

A.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 A.1.5.

~~The design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 timesmultiplied by the nominal flexural strength (M_n) of the member, except that the nominal shear strength (V_n) need not exceed 2.5 timesmultiplied by required shear strength (V_u).~~

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

A.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 A.1.8.3. 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.

A.1.5 *Strength-reduction factors*

A.1.5.1*Combinations of flexure and axial load in reinforced AAC masonry* — The value of ϕ shall be taken as 0.90 for reinforced AAC masonry designed to resist ~~for~~ flexure, axial load, or combinations thereof.

A.1.5.2*Combinations of flexure and axial load in unreinforced AAC masonry* — The value of ϕ shall be taken as 0.60 for unreinforced AAC masonry designed ~~for~~ to resist flexure, axial load, or combinations thereof.

A.1.5.3*Shear* — The value of ϕ shall be taken as 0.80 for AAC masonry designed ~~for~~ to resist shear.

A.1.5.4*Anchor bolts* — For cases where the nominal strength of an anchor bolt is controlled by AAC masonry breakout, ϕ shall be taken as 0.50. For cases where the nominal strength of an anchor bolt is controlled by anchor bolt steel, ϕ shall be taken as 0.90. For cases where the nominal strength of an anchor bolt is controlled by anchor pullout, ϕ shall be taken as 0.65.

A.1.5.5*Bearing* — For cases involving bearing on AAC masonry, ϕ shall be taken as 0.60.

A.1.6 *Deformation requirements*

~~**A.1.6.1***Drift limits* — Under loading combinations that include earthquake, masonry structures shall be designed so the calculated story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from the legally adopted building code. When the legally adopted building code does not provide allowable story drifts, structures shall be designed so the calculated story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from ASCE 7-02.~~

~~For determining drift, the calculated deflection shall be multiplied by C_d as indicated in the legally adopted building code. When the legally adopted building code does not provide C_d values, the deflection amplification factors shall be taken from ASCE 7-02.~~

~~**A.1.6.2***Deflection of unreinforced (plain) AAC masonry* — Deflection calculations for unreinforced (plain) AAC masonry members shall be based on uncracked section properties.

A.1.6.~~3~~2 *Deflection of reinforced AAC masonry* — Deflection calculations for reinforced AAC masonry members shall be based on cracked section properties including the reinforcement and grout. The flexural and shear stiffness properties assumed for deflection calculations shall not exceed one-half of the gross section properties unless a cracked-section analysis is performed.

A.1.7 *Anchor bolts*

Headed and bent-bar anchor bolts shall be embedded in grout, and shall be designed in accordance with Section 3.1.6 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.

A.1.8 *Material properties*

A.1.8.1*Compressive strength*

A.1.8.1.1 *Masonry compressive strength* — The specified compressive strength of AAC masonry, f'_{AAC}, shall equal or exceed 290 psi (3.45 MPa).

A.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).

A.1.8.2*Masonry splitting tensile strength* – The splitting tensile strength $f_{t\,AAC}$ shall be determined by Eq. A-1.

$$f_{t\,AAC} = 2.4\sqrt{f'_{AAC}} \qquad \text{(A-1)}$$

A.1.8.3*Masonry modulus of rupture* — The modulus of rupture, f_{rAAC}, for AAC masonry elements shall be taken as ~~two timesmultiplied by~~twice the masonry splitting tensile strength, f_{tAAC}. If a section of AAC masonry contains a horizontal leveling bed, the value of f_{rAAC} shall not exceed 50 psi (345 kPa) at that section. If a section of AAC contains a bed joint between thin-bed mortar and AAC, the value of f_{rAAC} shall not exceed 80 psi (552 kPa) at that section.

A.1.8.4*Masonry direct shear strength* – The direct shear strength, f_v, across an interface of AAC material shall be determined by Eq. A-2~~across an interface of AAC material~~, and shall be taken as 37 psi (255 kPa) across an interface between grout and AAC material.

$$f_v = 0.15 f'_{AAC} \qquad \text{(A-2)}$$

A.1.8.5*Coefficient of friction* – The coefficient of friction between AAC and AAC shall be 0.75. The coefficient of friction between AAC and thin-bed mortar or between AAC and leveling-bed mortar shall be 1.0.

A.1.8.6*Reinforcement strength* — Masonry design shall be based on a reinforcement strength equal to the specified yield strength of reinforcement, f_y, which shall not exceed 60,000 psi (413.7 MPa). The actual yield strength shall not exceed 1.3 ~~times~~multiplied by the specified yield strength. The compressive resistance of steel reinforcement shall be neglected, unless lateral reinforcement is provided in compliance with the requirements of Section ~~2.1.6.5~~1.14.1.3.

A.1.9 ~~*Section properties*~~

~~Member strength shall be computed using section properties based on the minimum net cross-sectional area of the member under consideration. Section properties shall be based on specified dimensions.~~

~~**A.1.10** *Concentrated loads*~~

A.1.~~10~~9.1 ~~For computing compressive capacity for walls laid in running bond, the effective cross-sectional area shall be taken as the product of the width of the bearing area times~~multiplied by ~~a length not to exceed the width of the bearing area plus four times~~multiplied by ~~the thickness of the supporting wall, or the center-to-center distance between concentrated loads.~~

~~**A.1.109.2** Bearing capacity shall be computed using a bearing area determined as follows:~~

~~(a)The direct bearing area A_1, or~~

~~(b) $A_1\sqrt{A_2 / A_1}$ but not more than $2A_1$, where A_2 is the supporting surface wider than A_1 on all sides, or A_2 is the area of the lower base of the largest frustum of a right pyramid or cone having A_1 as upper base sloping at 45 degrees from the horizontal and wholly contained within the support. For walls in other than running bond, area A_2 shall terminate at head joints.~~

A.1.109.3 Design bearing strength of AAC masonry shall equal $\phi f'_{AAC}$ multiplied by the bearing area, A_{br}, as defined in ~~A.1.109.2 (a) or A.1.109.2 (b)~~Section 1.9.6.5

A.1.~~10~~9.4 ~~—~~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 not less than:

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)

A.2 —Unreinforced (plain) AAC masonry

A.2.1 *Scope*

The requirements of Section A.2 are in addition to the requirements of Chapter 1 and Section A.1, and govern masonry design in which AAC masonry is used to resist tensile forces.

A.2.1.1*Strength for resisting loads* — Unreinforced (plain) AAC masonry members shall be designed using the strength of masonry units, mortar, and grout in resisting design loads.

A.2.1.2*Strength contribution from reinforcement* — Stresses in reinforcement shall not be considered effective in resisting design loads.

A.2.1.3*Design criteria* — Unreinforced (plain) AAC masonry members shall be designed to remain uncracked.

A.2.2 *Flexural strength of unreinforced (plain) AAC masonry members*

The following assumptions shall apply when determining the flexural strength of unreinforced (plain) AAC masonry members:

(a) Strength design of members for factored flexure and axial load shall be in accordance with principles of engineering mechanics.

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

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

(d) Flexural compressive stress in combination with axial compressive stress in masonry shall be assumed 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 A.1.8.3.

A.2.3 *Nominal axial strength of unreinforced (plain) AAC masonry members*

Nominal axial strength, P_n, shall be computed using Eq. (A-3) or Eq. (A-4).

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

$$P_n = 0.80\left\{0.85 A_n f'_{AAC}\left[1-\left(\frac{h}{140r}\right)^2\right]\right\} \qquad \text{(A-3)}$$

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

$$P_n = 0.80\left[0.85 A_n f'_{AAC}\left(\frac{70r}{h}\right)^2\right] \qquad \text{(A-4)}$$

A.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.

A.2.45 *Nominal shear strength of unreinforced (plain) AAC masonry members*

The nominal shear strength of AAC masonry, V_{nAAC}, shall be the least of the values computed by Sections A.3.4.1.2.1 through A.3.4.1.2.3. In evaluating nominal shear strength by Section A.3.4.1.2.3, effects of reinforcement shall be neglected. The provisions of A.3.4.1.2 shall apply to AAC shear walls laid in other than running bond.

A.2.56 *Flexural cracking*

The flexural cracking strength shall be computed in accordance with Section A.3.6.5.

A.3 — Reinforced AAC masonry

A.3.1 *Scope*

The requirements of this section are in addition to the requirements of Chapter 1 and Section A.1 and govern AAC masonry design in which reinforcement is used to resist tensile forces.

A.3.2 *Design assumptions*

The following assumptions apply to the design of reinforced AAC masonry:

(a) There is strain continuity between the reinforcement, grout, and AAC masonry such that applicable loads are resisted in a composite manner.

(b) The nominal strength of singly 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 0.003.

(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 s~~S~~tresses in reinforcement shall be ~~taken~~calculated as the product of steel modulus of elasticity, E_s, ~~times~~multiplied by and steel strain, ε_s, but shall not be greater than f_y.

(f) The tensile strength of AAC masonry shall be neglected in calculating flexural strength but shall be considered in calculating deflection.

(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 ~~zone~~ stress block bounded by edges of the cross section and a straight line ~~located~~ parallel to the neutral axis and located at a distance $a = 0.67\, c$ 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 ~~that~~ the neutral axis.

A.3.3 *Reinforcement requirements and details*

A.3.3.1*Reinforcing bar size limitations* — Reinforcing bars used in AAC masonry shall not be larger than No. 9 (M#29). The nominal bar diameter shall not exceed one-eighth of the nominal member thickness and shall not exceed one-quarter of the least clear dimension of the ~~cell, course, or collar joint~~grout space in which it is placed. In plastic hinge zones, the area of reinforcing bars placed in a ~~cell or in a course of hollow unit construction~~grout space shall not exceed 3 percent of the ~~cell~~ grout space area. In other than plastic hinge zones, the area of reinforcing bars placed in a ~~cell or in a course of hollow unit construction~~grout space shall not exceed 4.5 percent of the ~~cell~~ grout space area.

A.3.3.2 Standard hooks — The equivalent embedment length to develop standard hooks in tension, l_e, shall be determined by Eq. (A-5):

$$l_e = 13 d_b \qquad \text{(A-5)}$$

A.3.3.3*Development*

A.3.3.3.1 *Development of tension and compression reinforcement* — The required tension or compression reinforcement shall be developed in accordance with the following provisions:

The required development length of reinforcement shall be determined by Eq. (A-6), but shall not be less than 12 in. (305 mm).

$$l_d = \frac{0.13\, d_b^{\,2} f_y \gamma}{K_{AAC} \sqrt{f'_g}} \qquad \text{(A-6)}$$

K_{AAC} shall not exceed the least of the grout cover, the clear spacing between adjacent reinforcement, nor 5 ~~times~~multiplied by ~~by~~ d_b .

$\gamma = 1.0$ for No. 3 (M#10) through No. 5 (M#16) bars;

$\gamma = 1.3$ for No. 6 (M#19) through No. 7 (M#22) bars; and

$\gamma = 1.5$ for No. 8 (M#25) through No. 9 (M#29) bars.

A.3.3.3.2 *Development of shear reinforcement* — Shear reinforcement shall extend the depth of the member less cover distances.

A.3.3.3.2.1 Except at wall intersections, the end of a horizontal reinforcing bar needed to satisfy shear strength requirements of Section A.3.4.1.2, shall be bent around the edge vertical reinforcing bar with a 180-degree hook. The ends of single leg or U-stirrups shall be anchored by one of the following means:

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

(b) For No. 5 (M #16) bars and smaller, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of $l_d/3$. The $l_d/3$ embedment of a stirrup leg shall be taken as the distance between mid-depth of the member, $d/2$, and the start of the hook (point of tangency).

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

A.3.3.3.2.2 At wall intersections, horizontal reinforcing bars needed to satisfy shear strength requirements of Section A.3.4.1.2 shall be bent around the edge vertical reinforcing bar with a 90-degree standard hook and shall extend horizontally into the intersecting wall a minimum distance at least equal to the development length.

A.3.3.4*Splices* — Reinforcement splices shall comply with one of the following:

(a) The minimum length of lap for bars shall be 12 in. (305 mm) or the development length determined by Eq. (A-6), whichever is greater.

(b) A welded splice shall have the bars butted and welded to develop at least 125 percent of the yield strength, f_y , of the bar in tension or compression, as required.

(c) Mechanical splices shall have the bars connected to develop at least 125 percent of the yield strength, f_y , of the bar in tension or compression, as required.

A.3.3.5*Maximum reinforcement percentages* — The ratio of reinforcement, ρ, shall be calculated in accordance with Section 3.3.3.5 with the following exceptions:

The maximum usable strain, ε_{mu} , at the extreme masonry compression fiber shall be assumed to be 0.003 for AAC masonry.

The strength of the compression zone shall be calculated as 85 percent of f'_{AAC} multiplied by 67 percent of the area of the compression zone.

A.3.3.6*Bundling of reinforcing bars* — Reinforcing bars shall not be bundled.

A.3.4 *Design of beams, piers, and columns*

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

A.3.4.1*Nominal strength*

A.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 A.3.2 and A.3.4.1. For any value of nominal flexural strength, the corresponding nominal axial strength calculated in accordance with Sections A.3.2 and A.3.4.1 shall be modified for the effects of slenderness. 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 Eq. (A-7) or Eq. (A-8), as appropriate.

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

$$P_n = 0.80\left[0.85 f'_{AAC}\left(A_n - A_s\right) + f_y A_s\right]\left[1 - \left(\frac{h}{140r}\right)^2\right] \quad \text{(A-7)}$$

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

$$P_n = 0.80\left[0.85 f'_{AAC}\left(A_n - A_s\right) + f_y A_s\right]\left(\frac{70r}{h}\right)^2 \quad \text{(A-8)}$$

A.3.4.1.2 *Nominal shear strength* — Nominal shear strength, V_n, shall be computed using Eq. (A-9) and either Eq. (A-10) or Eq. (A-11), as appropriate.

$$V_n = V_{AAC} + V_s \quad V_n = V_{nAAC} + V_s \qquad \text{(A-9)}$$

where V_n shall not exceed the following:

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

$$V_n \le 6 A_n \sqrt{f'_{AAC}} \qquad \text{(A-10)}$$

(b) Where $M_u/V_u\, d_v \ge 1.00$

$$V_n \le 4 A_n \sqrt{f'_{AAC}} \qquad \text{(A-11)}$$

(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 interpolated.

The nominal masonry shear strength shall be taken as the least of the values computed using Section A.3.4.1.2.1 through A.3.4.1.2.3. Nominal shear strength provided by reinforcement, V_{ns}, shall include only deformed reinforcement embedded in grout for AAC shear walls.

A.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 computed using Eq. (A-12a) for AAC masonry with mortared head joints, and Eq. (A-12b) for masonry with unmortared head joints:

$$V_{AAC} = 0.95\, \ell_w\, t \sqrt{f'_{AAC}} \sqrt{1 + \frac{P_u}{2.4\sqrt{f'_{AAC}}\, \ell_w\, t}} \quad V_{nAAC} = 0.95\, \ell_w\, t \sqrt{f'_{AAC}} \sqrt{1 + \frac{P_u}{2.4\sqrt{f'_{AAC}}\, \ell_w\, t}} \qquad \text{(A-12a)}$$

$$V_{AAC} = 0.66\, \ell_w\, t \sqrt{f'_{AAC}} \sqrt{1 + \frac{P_u}{2.4\sqrt{f'_{AAC}}\, \ell_w\, t}} \quad V_{nAAC} = 0.66\, \ell_w\, t \sqrt{f'_{AAC}} \sqrt{1 + \frac{P_u}{2.4\sqrt{f'_{AAC}}\, \ell_w\, t}} \qquad \text{(A-12b)}$$

For AAC masonry in other than running bond, nominal masonry shear strength as governed by web-shear cracking, V_{nAAC}, shall be computed using Eq. (A-12c):

$$~~V_{AAC} = 0.9\sqrt{f'_{AAC}}\ A_n + 0.05P_u~~ \; V_{nAAC} = 0.9\sqrt{f'_{AAC}}\ A_n + 0.05P_u \qquad \text{(A-12c)}$$

A.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 computed as follows:

$$~~V_{AAC} = 0.17 f'_{AAC} t \frac{h \cdot l_w^{\,2}}{h^2 + (\frac{3}{4} l_w)^2}~~ \; V_{nAAC} = 0.17 f'_{AAC} t \frac{h \cdot l_w^{\,2}}{h^2 + (\frac{3}{4} l_w)^2} \qquad \text{(A-13a)}$$

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.

A.3.4.1.2.3 *Nominal shear strength as governed by sliding shear* — At an unbonded interface, nominal shear strength ~~as~~ governed by sliding shear, V_{nAAC}, shall be as follows:

$$~~V_{AAC} = \mu_{AAC} P_u~~ \; V_{nAAC} = \mu_{AAC} P_u \qquad \text{(A-13b)}$$

At an interface where thin-bed mortar or leveling-bed mortar are present, the nominal sliding shear capacity shall be calculated by Eq. A-13b using the coefficient of friction from Section A.1.8.5.

A.3.4.1.2.4 *Nominal shear strength provided by shear reinforcement* — Nominal shear strength provided by reinforcement, V_{ns}, shall be computed as follows:

$$V_{ns} = \left(\frac{A_v}{s}\right) f_y d_v \qquad \text{(A-14)}$$

A.3.4.1.2.5 Nominal shear strength ~~as~~ governed by out-of-plane loading shall be computed as follows:

$$~~V_{AAC} = 0.8\sqrt{f'_{AAC}}\ bd~~ \; V_{nAAC} = 0.8\sqrt{f'_{AAC}}\ bd \qquad \text{(A-15)}$$

A.3.4.2*Beams* — Design of beams shall meet the requirements of Section 1.13 and the additional requirements of Section A.3.4.2.

A.3.4.2.1 Members designed primarily to resist flexure shall comply with the requirements of Section A.3.4.2. The factored axial compressive force on a beam shall not exceed $0.05 A_n f'_{AAC}$.

A.3.4.2.2 *Longitudinal reinforcement*

A.3.4.2.2.1 The variation in longitudinal reinforcing bars shall not be greater than one bar size. Not more than two bar sizes shall be used in a beam.

A.3.4.2.2.2 The nominal flexural strength of a beam shall not be less than 1.3 ~~times~~multiplied by the nominal cracking moment ~~strength~~ of the beam, M_{cr}. The modulus of rupture, f_{rAAC}, for this calculation shall be determined in accordance with Section A.1.8.3.

A.3.4.2.3 *Transverse reinforcement* — Transverse reinforcement shall be provided where V_u exceeds ϕV_{nAAC}. The factored shear, V_u, shall include the effects of lateral load. When transverse reinforcement is required, the following provisions shall apply:

(a) Transverse reinforcement shall be a single bar with a 180-degree hook at each end.

(b) Transverse reinforcement shall be hooked around the longitudinal reinforcement.

(c) The minimum area of transverse reinforcement shall be $0.0007 bd_v$.

(d) The first transverse bar shall not be located more than one-fourth of the beam depth, d_v, from the end of the beam.

(e) The maximum spacing shall not exceed the lesser of one-half the depth of the beam or 48 in. (1219 mm).

A.3.4.2.4 *Construction* — Beams shall be grouted solid.

A.3.4.2.5 Dimensional limits ~~— Dimensions shall be in accordance with the following:~~

~~(a) The clear distance between locations of lateral bracing of the compression side of the beam shall not exceed 32 times the least width of the compression area.~~

~~(b)~~ The nominal depth of a beam shall not be less than 8 in. (203 mm).

A.3.4.3 *Piers*

A.3.4.3.1 The factored axial compression force on the piers shall not exceed $0.3 A_n f'_{AAC}$.

A.3.4.3.2 *Longitudinal reinforcement* — A pier subjected to in-plane stress reversals

shall be reinforced symmetrically about the geometric center of the pier. The longitudinal reinforcement of piers shall comply with the following:

(a) At least o~~O~~ne bar shall be provided in each~~the~~ end cell~~s~~.

(b) The minimum area of longitudinal reinforcement shall be 0.0007 *bd*.

~~(c) Longitudinal reinforcement shall be uniformly distributed throughout the depth of the element.~~

A.3.4.3.3 *Dimensional limits* — Dimensions shall be in accordance with the following:

(a) The nominal thickness of a pier shall not be less than 6 in. (152 mm) and shall not exceed 16 in. (406 mm).

(b) The distance between lateral supports of a pier shall not exceed 25 ~~times~~multiplied by the nominal thickness of a pier except as provided for in Section A.3.4.3.3(c).

(c) When the distance between lateral supports of a pier exceeds 25 ~~times~~multiplied by the nominal thickness of the pier, design shall be based on the provisions of Section A.3.5.

(d) The nominal length of a pier shall not be less than three ~~times~~multiplied by its nominal thickness nor greater than six ~~times~~multiplied by its nominal thickness. The clear height of a pier shall not exceed five ~~times~~multiplied by its nominal length.

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

A.3.4.4*Columns* — Design of columns shall meet the requirements of Section 1.14 and the additional requirements of Section A.3.4.4.

~~**A.3.4.4.1** *Longitudinal reinforcement — Longitudinal reinforcement shall be a minimum of four bars, one in each corner of the column, and shall comply with the following:*~~

~~(b)*Maximum reinforcement area shall be determined in accordance with Section A.3.3.5, but shall not exceed 0.04 A_n.*~~

~~(c)*Minimum reinforcement area shall be 0.0025 A_n.*~~

~~*Longitudinal reinforcement shall be uniformly distributed throughout the depth of the element.*~~

~~**A.3.4.4.2** *Lateral ties — Lateral ties shall be provided in accordance with Section 2.1.6.5.*~~

~~**A.3.4.4.3**~~**2** *Construction* — Columns shall be solid grouted.

A.3.4.4.2 *Dimensional limits* — Dimensions shall be in accordance with the following:

(a) ~~The nominal width of a column shall not be less than 8 in. (203 mm).~~

~~(b)~~ The distance between lateral supports of a column shall not exceed 30 ~~times~~multiplied by its nominal width.

(~~a~~b) The nominal depth of a column shall not be less than 8 in. (203 mm) and not greater than three ~~times~~multiplied by its nominal width.

A.3.5 *Wall design for out-of-plane loads*

A.3.5.1 ~~*General*~~ *Scope* — The requirements of Section A.3.5 are for the design of walls for out-of-plane loads.

~~**A.3.5.2** *Effective compressive width per bar* — The effective compressive width per bar shall be determined in accordance with Section 1.9.6.~~

~~**A.3.5.23**~~ *Maximum reinforcement* — The maximum reinforcement ratio shall be determined by Section A.3.3.5.

A.3.5.3~~.3~~4 *Moment and deflection calculations* — Moment and deflection calculations in Section A.3.5.4~~5~~4 and A.3.5.6~~5~~ 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.

A.3.5.4~~5~~4 *Walls with factored axial stress of 0.20 f'_{AAC} or less* — The procedures set forth in this section shall be used when the factored axial load stress at the location of maximum moment satisfies the requirement computed by Eq. (A-16).

$$\left(\frac{P_u}{A_g}\right) \le 0.20 f'_{AAC} \qquad \text{(A-16)}$$

When the slenderness ratio exceeds 30, the factored axial stress shall not exceed 0.05 f'_{AAC}

Factored moment and axial force shall be determined at the midheight of the wall and shall be used for design. The factored moment, M_u, at the midheight of the wall shall be computed using Eq. (A-17).

$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u \qquad \text{(A-17)}$$

Where:

$$P_u = P_{uw} + P_{uf} \qquad \text{(A-18)}$$

The deflection due to factored loads (δ_u) shall be obtained using Eq. (A-23) and (A-24) and

replacing M_{ser} with M_u and δ_s with δ_u.

The design strength for out-of-plane wall loading shall be in accordance with Eq. (A-19).

$$M_u \le \phi M_n \qquad \text{(A-19)}$$

~~Where:~~The nominal moment shall be calculated using Eqs. (A-20) and (A-21) if the reinforcing steel is placed in the center of the wall.

$$M_n = \left(A_s f_y + P_u\right)\left(d - \frac{a}{2}\right) \qquad \text{(A-20)}$$

$$a = \frac{\left(P_u + A_s f_y\right)}{0.85 f'_{AAC}\, b} \qquad \text{(A-21)}$$

The nominal shear strength for out-of-plane loads shall be determined by Section A.3.4.1.2.5.

~~**A.3.5.5** *Walls with factored axial stress greater than 0.2 f'_{AAC} or slenderness ratios greater than 30* — Such walls shall be designed in accordance with the provisions of Section A.3.5.4 and shall have a minimum nominal thickness of 6 in. (152 mm).~~

~~The nominal shear strength shall be determined by Section A.3.4.1.2.5.~~

—**A.3.5.~~6~~5** *Deflection design* — The horizontal midheight deflection, δ_s, under service lateral and service axial loads (without load factors) shall be limited by the relation:

$$\delta_s \le 0.007\, h \qquad \text{(A-22)}$$

P-delta effects shall be included in deflection calculation. The midheight deflection shall be computed using either Eq. (A-23) or Eq. (A-24), as applicable.

(a) Where $M_{ser} < M_{cr}$

$$\delta_s = \frac{5 M_{ser} h^2}{48 E_{AAC} I_g} \qquad \text{(A-23)}$$

(b) Where $M_{cr} < M_{ser} < M_n$

$$\delta_s = \frac{5M_{cr}h^2}{48E_{AAC}I_g} + \frac{5\left(M_{ser} - M_{cr}\right)h^2}{48E_{AAC}I_{cr}} \qquad \text{(A-24)}$$

The cracking moment ~~strength~~ of the wall shall be computed using Eq. (A-25), where f_{rAAC} is given by Section A.1.8.3:

$$M_{cr} = S_n\left(f_{rAAC} + \frac{P}{A_n}\right) \qquad \text{(A-25)}$$

If the section of AAC masonry contains a horizontal leveling bed, the value of f_{rAAC} shall not exceed 60 psi (414 kPa).

A.3.6 *Wall design for in-plane loads*

A.3.6.1*Scope* — The requirements of Section A.3.6 are for the design of walls to resist in-plane loads.

A.3.6.2*Reinforcement* — Reinforcement shall be in accordance with the following:

(a) The amount of vertical reinforcement shall not be less than one-half the horizontal reinforcement.

(b) The maximum reinforcement ratio shall be determined in accordance with Section A.3.3.5.

A.3.6.3*Flexural and axial strength* — The nominal flexural and axial strength shall be determined in accordance with Section A.3.4.1.1.

A.3.6.4*Shear strength* — The nominal shear strength shall be computed in accordance with Section A.3.4.1.2.

A.3.6.5*Flexural cracking strength* — The flexural cracking strength shall be computed in accordance with Eq. (A-26), where f_{rAAC} is given by Section A.1.8.3:

$$V_{cr} = \frac{S_n}{h}\left(f_{rAAC} + \frac{P}{A_n}\right) \qquad \text{(A-26)}$$

If the section of AAC masonry contains a horizontal leveling bed, the value of f_{rAAC} shall not exceed 60 psi (414 kPa).

A.3.6.6 The maximum reinforcement requirements of Section A.3.3.5 shall not apply if a shear wall is designed to satisfy the requirements of Sections A.3.6.7 through A.3.6.10.

A.3.6.7 The need for special boundary elements at the edges of shear walls shall be evaluated in accordance with Section A.3.6.8 or A.3.6.9. The requirements of Section A.3.6.10 shall also be satisfied.

A.3.6.8 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 A.3.6.9.

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

$$c \geq \frac{l_w}{600\left(C_d \delta_{ne} / h_w\right)}$$

and c is calculated for the P_u given by ASCE 7~~-02~~ Load Combination 5 ($1.2D + 1.0E + L + 0.2S$) 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 5 is reducible to 0.5, as per exceptions to Section 2.3.2 of ASCE 7~~-02~~.

(b) Where special boundary elements are required by Section A.3.6.8 (a), the special boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of l_w or $M_u/4V_u$.

A.3.6.9 Shear walls not designed to the provisions of Section A.3.6.8 shall have special boundary elements at boundaries and edges around openings in shear walls where the maximum extreme fiber compressive stress, corresponding to factored forces including earthquake effect, exceeds $0.2f'_{AAC}$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15f'_{AAC}$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in Section 1.9.4.2.3 shall be used.

A.3.6.10 Where special boundary elements are required by Section A.3.6.8 or A.3.6.9, (a) through (d) shall be satisfied and tests shall be performed to verify the strain capacity of the element:

(a) The special boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of (c - 0.1l_w) and c/2.

(b) In flanged sections, the special boundary element shall include the effective flange width in compression and shall extend at least 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.

CONVERSION OF INCH-POUND UNITS TO SI UNITS

The equations in this Code are for use with the specified inch-pound units only. The equivalent units for use with SI units follow.

Code Eq. No. or Sec. No.	SI Unit Equivalent Equation	Units
1.8.2.2.1	$E_m = 700\,f'_m$ for clay masonry $E_m = 900\,f'_m$ for concrete masonry	f'_m in MPa
1.8.2.3.1	$E_{AAC} = 887.8\,(f'_{AAC})^{0.6}$	f'_{AAC} in MPa
1.8.2.4	$500\,f'_g$	f'_g in MPa
(1-1)	$I_{eff} = I_n\left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] \le I_n$	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
(1-2)	$A_{pt} = \pi l_b^2$	A_{pt} in mm^2 l_b in mm
(1-3)	$A_{pv} = \frac{\pi l_{be}^2}{2}$	A_{pv} in mm^2 l_{be} in mm
(2-1)	$B_{ab} = 0.11 A_{pt}\sqrt{f'_m}$ ~~$B_a = 0.042 A_p\sqrt{f'_m}$~~	A_{pt} in mm^2 B_{ab} in Newtons $\sqrt{f'_m}$ in MPa
(2-2)	$B_{as} = 0.6 A_b f_y$ ~~$B_a = 0.2 A_b f_y$~~	A_b in mm^2 B_{as} in Newtons f_y in MPa
(2-3)	$B_{ab} = 0.11 A_{pt}\sqrt{f'_m}$ ~~$A_p = \pi l_b^2$~~	A_{pt} in mm^2 B_{ab} in Newtons $\sqrt{f'_m}$ in MPa ~~A_p in mm^2~~ ~~l_b in mm~~
(2-4)	$B_{ap} = \frac{0.6\,f'_m e_b\,d_b + 120\pi(l_b + e_b + d_b)d_b}{\text{~~}A_p = \pi l_{be}^2\text{~~}}$	f'_m in MPa e_b in mm d_b in mm l_b in mm B_{ap} in Newtons ~~A_p in mm^2~~ ~~l_{be} in mm~~
(2-5)	$B_{as} = 0.6 A_b f_y$ ~~$B_v = 1072\sqrt[4]{f'_m A_b}$~~	A_b in mm^2 B_{as} in Newtons f_y in MPa ~~$\sqrt{f'_m}$ in MPa~~
(2-6)	$B_{ab} = 0.11 A_{pt}\sqrt{f'_m}$ ~~$B_v = 0.12 A_b f_y$~~	A_{pt} in mm^2 B_{ab} in Newtons $\sqrt{f'_m}$ in MPa ~~A_b in mm^2~~ ~~B_v in Newtons~~ ~~f_y in MPa~~
(2-7)	$B_{ab} = 1072\sqrt[4]{f'_m A_b}\,\frac{b_a}{B_a} + \frac{b_v}{B_v} \le 1$	B_{ab} in Newtons $\sqrt[4]{f'_m A_b}$ ~~B_v~~ in Newtons ~~b_a in Newtons~~ ~~b_v in Newtons~~

Code Eq. No. or Sec. No.	SI Unit Equivalent Equation	Units
(2-8)	$B_{vpry} = 2.0B_{ab} = 0.21A_{pt}\sqrt{f'_m}$	A_{pt} in mm^2 B_{ab} in Newtons B_{vpry} in Newtons $\sqrt{f'_m}$ in MPa
(2-9)	$B_{vs} = 0.36A_b f_y$	A_b in mm^2 B_{as} in Newtons f_y in MPa
(2-10)	$\frac{b_a}{B_a} + \frac{b_v}{B_v} \le 1$	b_a in Newtons b_v in Newtons B_a in Newtons B_v in Newtons
2.1.5.2.2(c)	$0.083\sqrt{unit\ compressive\ strength\ of\ header}$	$\sqrt{unit\ compressive\ strength\ of\ header}$ in MPa
(2-811)	$l_d = 0.22d_b F_s$	d_b in mm F_s in MPa l_d in mm
2.1.10.4.1.5(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
(2-912)	$l_d = \frac{1.5d_b^2 f_y \gamma}{K\sqrt{f'_m}}$	d_b in mm $\sqrt{f'_m}$ in MPa f_y in MPa K in mm l_d in mm
(2-1310)	$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$	F_a in MPa F_b in MPa f_a in MPa f_b in MPa
(2-1114)	$P \le (1/4)P_e$	P in Newtons P_e in Newtons
(2-1215)	$F_a = (1/4)f'_m\left[1-\left(\frac{h}{140r}\right)^2\right]$	F_a in MPa f'_m in MPa h in mm r in mm
(2-1316)	$F_a = (1/4)f'_m\left(\frac{70r}{h}\right)^2$	F_a in MPa f'_m in MPa h in mm r in mm
(2-1417)	$F_b = (1/3)f'_m$	F_b in MPa f'_m in MPa
(2-1518)	$P_e = \frac{\pi^2 E_m I_n}{h^2}\left(1-0.577\frac{e}{r}\right)^3$	E_m in MPa e in mm h in mm I in mm^4 P_e in Newtons r in mm

Code Eq. No. or Sec. No.	SI Unit Equivalent Equation	Units
(2-~~16~~19)	$f_v = \frac{VQ}{I_n b}$	b in mm f_v in MPa I in mm^4 Q in mm^3 V in Newtons
2.2.5.2(a)	$0.125\sqrt{f'_m}$	$\sqrt{f'_m}$ in MPa Answer in MPa
2.2.5.2(c)	$255 + 0.45\ N_v/A_n$	A_n in mm^2 N_V in Newtons Answer in kPa
2.2.5.2(d)	$255\ + 0.45\ N_v/A_n$	A_n in mm^2 N_V in Newtons Answer in kPa
2.2.5.2(e)	$414 + 0.45\ N_v/A_n$	A_n in mm^2 N_V in Newtons Answer in kPa
(2-~~17~~20)	$P_a = (0.25f'_m A_n + 0.65A_{st}F_s)\left[1-\left(\frac{h}{140r}\right)^2\right]$	A_n in mm^2 h in mm A_{st} in mm^2 P_a in Newtons F_s in MPa r in mm f'_m in MPa
(2-~~18~~21)	$P_a = (0.25f'_{\mathrm{m}}A_n + 0.65A_{\mathrm{st}}F_s)\left(\frac{70r}{h}\right)^2$	A_n in mm^2 h in mm A_{st} in mm^2 P_a in Newtons F_s in MPa r in mm f'_m in MPa
(2-~~19~~22)	$\rho_{\max} = \frac{nf'_m}{2f_y\left(n+\frac{f_y}{f'_m}\right)}$	f_y in MPa f'_m in MPa
(2-~~19~~2023)	$f_v = \frac{V}{bd}$	b in mm d in mm f_v in MPa V in Newtons
(2-~~20~~2124)	$F_v = 0.083\sqrt{f'_m}$	F_v in MPa $\sqrt{f'_m}$ in MPa
(2-~~21~~2225)	$F_v = 0.028[4-(M/Vd)]\sqrt{f'_m}$ but shall not exceed $0.55 - 0.31\ (M/Vd)$ in MPa	d in mm F_v in MPa M in Newton-mm V in Newtons $\sqrt{f'_m}$ in MPa
(2-~~22~~2326)	$F_v = 0.083\sqrt{f'_m}$	F_v in MPa $\sqrt{f'_m}$ in MPa
(2-~~2~~~~23~~27)	$F_v = 0.25\sqrt{f'_m}$	F_v in MPa $\sqrt{f'_m}$ in MPa

Code Eq. No. or Sec. No.	SI Unit Equivalent Equation	Units
(2-24528)	$F_v = 0.042[4-(M/Vd)]\sqrt{f'_m}$ but shall not exceed $0.82 - 0.31\ (M/Vd)$ in MPa	d in mm F_v in MPa M in Newton-mm V in Newtons $\sqrt{f'_m}$ in MPa
(2-25629)	$F_v = 0.125\sqrt{f'_m}$	F_v in MPa $\sqrt{f'_m}$ in MPa
(2-2672-30)	$A_v = \frac{Vs}{F_s d}$	A_v in mm^2 d in mm F_s in MPa s in mm V in Newtons
(3-1) & (3-4)	$B_{anb} = 0.33A_{pt}\sqrt{f'_m}$ ~~$B_{an} = 0.332A_{pt}\sqrt{f'_m}$~~	A_{pt} in mm^2 $\sqrt{f'_m}$ in MPa B_{anb} in Newtons
(3-2) & (3-5)	$B_{ans} = A_b f_y$	A_b in mm^2 f_y in MPa B_{ans} in Newtons
(3-3) & (3-7)	$A_{pt} = \pi\, l_b^{\,2}$ $B_{anb} = 0.33A_{pt}\sqrt{f'_m}$	A_{pt} in mm^2 $\sqrt{f'_m}$ in MPa B_{anb} in Newtons ~~l_b in mm~~ ~~A_{pt} in mm^2~~
(3-64)	$B_{anp} = 1.5f'_m e_b d_b + 300\pi(l_b + e_b + d_b)d_b$ ~~$B_{an} = 1.5f'_m e_b d_b + [3.07\pi(l_b + e_b + d_b)d_b]$~~	f'_m in MPa e_b in mm d_b in mm l_b in mm B_{anp} in Newtons
(3-5)	$B_{ans} = A_b f_y$	A_b in mm^2 f_y in MPa B_{ans} in Newtons
(3-6)	$B_{anb} = 0.33A_{pt}\sqrt{f'_m}$	A_{pt} in mm^2 $\sqrt{f'_m}$ in MPa B_{anb} in Newtons
(3-7)	$B_{vnc} = 3216\sqrt[4]{f'_m\ A_b}$	B_{ab} in Newtons $\sqrt[4]{f'_m A_b}$ in Newtons
(3-8)	$B_{vpry} = 2.0B_{ab} = 0.67A_{pt}\sqrt{f'_m}$ ~~$B_{vn} = 0.332A_{pv}\sqrt{f'_m}$~~	A_{pt} in mm^2 $\sqrt{f'_m}$ in MPa B_{vpry} ~~$_n$~~ in Newtons
(3-9)	$B_{vns} = 0.6A_b f_y$ ~~$B_{vn} = 0.6A_b f_y$~~	A_b in mm^2 f_y in MPa B_{vns} in Newtons
~~(3-10)~~	~~$A_{pv} = \frac{\pi\, l_{be}^{\,2}}{2}$~~	~~l_{be} in mm~~ ~~A_{pv} in mm^2~~

Code Eq. No. or Sec. No.	SI Unit Equivalent Equation	Units
(3-~~11~~10)	$\frac{b_{af}}{\phi B_{an}} + \frac{b_{vf}}{\phi B_{vn}} \le 1$ ~~$\frac{b_{af}}{\phi B_{an}} + \frac{b_{vf}}{\phi B_{vn}} \le 1$~~	b_{af} in Newtons b_{vf} in Newtons B_{an} in Newtons B_{vn} in Newtons
(3-~~12~~11)	$P_n = 0.80\left\{0.80 A_n f'_m\left[1-\left(\frac{h}{140r}\right)^2\right]\right\}$ For $\frac{h}{r} \le 99$	P_n in Newtons A_n in mm^2 f'_m in MPa
(3-1~~3~~2)	$P_n = 0.80\left(0.80 A_n f'_m\left(\frac{70r}{h}\right)^2\right)$ For $\frac{h}{r} > 99$	P_n in Newtons A_n in mm^2 f'_m in MPa h in mm r in mm
(3-1~~4~~3)	$M_c = \delta M_u$	M_c in N-mm M_{cu} in N-mm
(3-1~~5~~4)	$\delta = \frac{1}{1-\frac{P_u}{A_n f'_m\left(\frac{70r}{h}\right)^2}}$	A_n in mm^2 f'_m in MPa P_u in Newtons h in mm r in mm
3.2.4(a)	$0.375 A_n \sqrt{f'_m}$	A_n in mm^2 f'_m in MPa
3.2.4(b)	$0.83 A_n$	A_n in mm^2
3.2.4(c)	$0.26 A_n + 0.45 N_u$	A_n in mm^2 N_v in Newtons
3.2.4(d)	$0.26 A_n + 0.45 N_u$	A_n in mm^2 N_v in Newtons
3.2.4(e)	$0.414 A_n + 0.45 N_u$	A_n in mm^2 N_v in Newtons
3.2.4(f)	$0.103 A_n$	A_n in mm^2
~~(3-14)~~	~~$M_c = \delta M_u$~~	~~M_c in N-mm~~ ~~M_{cu} in N-mm~~
~~(3-15)~~	~~$\delta = \frac{1}{1-\frac{P_u}{A_n f'_m\left(\frac{70r}{h}\right)^2}}$~~	~~A_n in mm^2~~ ~~f'_m in MPa~~ ~~P_u in Newtons~~ ~~h in mm~~ ~~r in mm~~
(3-~~14~~3-1~~65~~)	$l_e = 13 d_b$	l_e in mm d_b in mm
(3-~~15~~3-1~~76~~)	$l_d = \frac{1.5 d_b^2 f_y \gamma}{K\sqrt{f'_m}}$	d_b in mm $\sqrt{f'_m}$ in MPa f_y in MPa K in mm l_d in mm

Code Eq. No. or Sec. No.	SI Unit Equivalent Equation	Units
(3-163-187)	~~$P_n = 0.80\left[0.80 f'_m (A_n - A_s) + f_y A_s\right]\left[1 - \left(\frac{h}{140r}\right)^2\right]$~~ $P_n = 0.80\left[0.80 f'_m (A_n - A_{st}) + f_y A_{st}\right]\left[1 - \left(\frac{h}{140r}\right)^2\right]$	A_e in mm^2 A_s in mm^2 f'_m in MPa f_y in MPa P_n in Newtons h in mm r in mm
(3-173-198)	$P_n = 0.80\left[0.80 f'_m (A_n - A_{st}) + f_y A_{st}\right]\left(\frac{70r}{h}\right)^2$ ~~$P_n = 0.80\left[0.80 f'_m (A_n - A_s) + f_y A_s\right]\left(\frac{70r}{h}\right)^2$~~	A_e in mm^2 A_s in mm^2 f'_m in MPa f_y in MPa P_n in Newtons h in mm r in mm
(3-183-2019)	$V_n = V_{nm} + V_{ns}$	V_{nm} in Newtons V_{ns} in Newtons V_n in Newtons
(3-193-2120)	$V_n \le 6 A_n \sqrt{f'_m}$ For $\frac{M_u}{V_u d_v} \le 0.25$	A_n in mm^2 M_u in N-mm V_u in Newtons d_v in mm V_n in Newtons $\sqrt{f'_m}$ in MPa
(3-203-2221)	$V_n \le 4 A_n \sqrt{f'_m}$ For $\frac{M_u}{V_u d_v} \ge 1.00$	A_n in mm^2 M_u in N-mm V_u in Newtons d_v in mm V_n in Newtons $\sqrt{f'_m}$ in MPa
(3-213-2322)	~~$V_m = 0.83\left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25P$~~ $V_{nm} = 0.083\left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25P$	A_n in mm^2 M_u in N-mm V_u in Newtons d_v in mm V_{nm} in Newtons $\sqrt{f'_m}$ in MPa
(3-223-2423)	$V_{ns} = 0.5\left(\frac{A_v}{s}\right) f_y\, d_v$	A_v in mm^2 f_y in MPa d_v in mm s in mm V_{ns} in Newtons
(3-233-2524)	$\left(\frac{P_u}{A_g}\right) \le 0.05 f'_m$	P_u in Newtons A_g in mm^2 f'_m in MPa

Code Eq. No. or Sec. No.	SI Unit Equivalent Equation	Units
(~~3-24~~3-~~26~~5)	$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u$	h in mm w_u in N/mm P_{uf} in Newtons e_u in mm P_u in Newtons ε_u in mm δ_u in mm M_u in N-mm
(~~3-25~~3-~~27~~6)	$P_u = P_{uw} + P_{uf}$	P_u in Newtons P_{uf} in Newtons P_{uw} in Newtons
(~~3-26~~3-~~27~~8)	$M_u \le \phi M_n$	M_u in N-mm M_n in N-mm
(~~3-27~~3-~~28~~9)	$M_n = \left(A_s f_y + P_u\right)\left(d - \frac{a}{2}\right)$	M_n in N-mm A_s in mm^2 f_y in MPa d in mm a in mm P_u in Newtons
(~~3-28~~3-~~29~~30)	$a = \frac{\left(P_u + A_s f_y\right)}{0.80 f'_m b}$	a in mm f'_m in MPa A_s in mm^2 P_u in Newtons f_y in MPa b in mm
(~~3-29~~3-~~30~~1)	$\delta_s \le 0.007\, h$	$\delta\epsilon_s$ in mm h in mm
(~~3-30~~3-~~31~~2)	$\delta_s = \frac{5 M_{ser} h^2}{48 E_m I_g}$ For $M_{ser} \le M_{cr}$	$\delta\epsilon_s$ in mm h in mm E_m in MPa I_g in mm^4 M_{ser} in N-mm M_{cr} in N-mm
(~~3-31~~3-~~32~~3)	$\delta_s = \frac{5 M_{cr} h^2}{48 E_m I_g} + \frac{5\left(M_{ser} - M_{cr}\right) h^2}{48 E_m I_{cr}}$ For $M_{cr} < M_{ser} < M_n$	$\delta\epsilon_s$ in mm h in mm E_m in MPa I_g in mm^4 M_{ser} in N-mm M_{cr} in N-mm M_n in N-mm I_{cr} in mm^4
3.3.6.6	$P_u \le 0.10 A_g f'_m$ $P_u \le 0.05 A_g f'_m$	P_u in Newtons A_g in mm^2 f'_m in MPa
3.3.6.6	$\frac{M_u}{V_u l_w} \le 1.0$	M_u in N-mm V_u in Newtons l_w in mm
3.3.6.6	$V_u \le 0.25 A_n \sqrt{f'_m}$ and $\frac{M_u}{V_u l_w} \le 3.0$	A_n in mm^2 f'_m in MPa l_w in mm M_u in N-mm V_u in Newtons

Code Eq. No. or Sec. No.	SI Unit Equivalent Equation	Units
3.3.6.8 (a)	$c \geq \frac{l_w}{600\left(C_d \delta_{ne} / h_w\right)}$	c in mm h_w in mm l_w in mm δ_{ne} in mm
(4-1)	$a = \frac{f_{ps} A_{ps} + f_y A_s + P_u}{0.85 f'_m b}$	a in mm f_{ps} in MPa A_{ps} in mm^2 f_y in MPa A_s in mm^2 P_u in Newtons f'_m in MPa b in mm
(4-2)	$M_n = (f_{ps} A_{ps} + f_y A_s + P_u)(d - \frac{a}{2})$	M_n in Newton·mm f_{ps} in MPa A_{ps} in mm^2 f_y in MPa A_s in mm^2 P_u in Newtons d in mm a in mm
(4-3)	$f_{ps} = f_{se} + (6{,}900)\left(\frac{d}{l_p}\right)\sqrt{1 - 1.4\left(\frac{f_{pu} A_{ps}}{b d f'_m}\right)}$	f_{ps} in MPa f_{se} in MPa d in mm l_p in mm f_{pu} in MPa A_{ps} in mm^2 b in mm f'_m in MPa
(4-4)	$f_{ps} = f_{se} + (4{,}830)\left(\frac{d}{l_p}\right)\sqrt{1 - 1.4\left(\frac{f_{pu} A_{ps}}{b d f'_m}\right)}$	f_{ps} in MPa f_{se} in MPa d in mm l_p in mm f_{pu} in MPa A_{ps} in mm^2 b in mm f'_m in MPa
(A-1)	$f_t = 0.2\sqrt{f'_{AAC}}$	f_t in MPa $\sqrt{f'_{AAC}}$ in MPa
(A-2)	$f_v = 0.15 f'_{AAC}$	f_v in MPa f'_{AAC} in MPa
(A-3)	$P_n = 0.80\left[0.85 f'_{AAC}(A_n - A_s) + f_y A_s\left[1 - \left(\frac{h}{140r}\right)^2\right]\right]$	h in mm r in mm A_n in mm^2 A_s in mm^2 f_y in MPa f'_{AAC} in MPa P_n in Newtons

Code Eq. No. or Sec. No.	SI Unit Equivalent Equation	Units
(A-4)	$P_n = 0.80\left[0.85 f'_{AAC}(A_n - A_s) + f_y A_s\left(\frac{70r}{h}\right)^2\right]$	h in mm r in mm A_n in mm^2 A_s in mm^2 f_y in MPa f'_{AAC} in MPa P_n in Newtons
(A-5)	$l_e = 13 d_b$	l_e in mm d_b in mm
(A-6)	$l_d = \frac{1.5 d_b^{\,2} f_y \gamma}{K_{AAC}\sqrt{f'_g}}$	l_d, in mm d_b in mm K_{AAC} in mm f_y in MPa $\sqrt{f'_g}$ in MPa
(A-7)	$P_n = 0.80\left[0.85 f'_{AAC}(A_n - A_s) + f_y A_s\left[1 - \left(\frac{h}{140r}\right)^2\right]\right]$	h in mm r in mm A_n in mm^2 A_s in mm^2 f_y in MPa f'_{AAC} in MPa P_n in Newtons
(A-8)	$P_n = 0.80\left[0.85 f'_{AAC}(A_n - A_s) + f_y A_s\left(\frac{70r}{h}\right)^2\right]$	h in mm r in mm A_n in mm^2 A_s in mm^2 f_y in MPa f'_{AAC} in MPa P_n in Newtons
(A-9)	~~$V_n = V_{AAC} + V_{ns}$~~ $V_n = V_{nAAC} + V_{ns}$	$V_n = V_{nAAC} + V_{ns}$ in Newtons V_{nAAC} in Newtons V_{ns} in Newtons
(A-10)	$V_n \le 0.0008 A_n \sqrt{f'_{AAC}}$	V_n in Newtons $\sqrt{f'_{AAC}}$ in MPa A_n in mm^2
(A-11)	$V_n \le 0.0005 A_n \sqrt{f'_{AAC}}$	V_n in Newtons $\sqrt{f'_{AAC}}$ in MPa A_n in mm^2
(A-12a)	$V_{nAAC} = 0.08\,\ell_w\, t\,\sqrt{f'_{AAC}}\,\sqrt{1 + \frac{P_u}{0.2\sqrt{f'_{AAC}}\,\ell_w\, t}}$	V_{nAAC} in Newtons P_u in Newtons $\sqrt{f'_{AAC}}$ in MPa l_w in m t in m
(A-12b)	$V_{nAAC} = 0.055\,\ell_w\, t\,\sqrt{f'_{AAC}}\,\sqrt{1 + \frac{P_u}{0.2\sqrt{f'_{AAC}}\,\ell_w\, t}}$	V_{nAAC} in Newtons P_u in Newtons $\sqrt{f'_{AAC}}$ in MPa l_w in m t in m

Code Eq. No. or Sec. No.	SI Unit Equivalent Equation	Units
(A-12c)	$V_{nAAC} = 0.075\sqrt{f'_{AAC}}\ A_n + 0.05P_u$	V_{nAAC} in Newtons P_u in Newtons $\sqrt{f'_{AAC}}$ in MPa A_n in m^2
(A-13a)	$V_{nAAC} = 170000\ f'_{AAC}\ t\left[\frac{h\,(l_w)^2}{h^2 + \left(\frac{3l_w}{4}\right)^2}\right]$	V_{nAAC} in Newtons f'_{AAC} in MPa t in m in m h in m l_w in m
(A-13b)	$V_{nAAC} = \mu_{AAC} P_u$	V_{nAAC} in Newtons P_u in Newtons
(A-14)	$V_{ns} = \left(\frac{A_v}{s}\right) f_y d_v$	V_{ns} in Newtons f_y in MPa s in mm d_v in mm A_v in mm^2
(A-15)	$V_{nAAC} = 66500\ \sqrt{f'_{AAC}}\ bd$	V_{nAAC} in Newtons $\sqrt{f'_{AAC}}$ in MPa b in m d in m
(A-16)	$\frac{P_u}{A_g} \le 0.2 f'_{AAC}$	P_u in Newtons f'_{AAC} in MPa A_g in mm^2
(A-17)	$M_u = \frac{w_u h^2}{8} + P_{uf}\ \frac{e_u}{2} + P_u \delta_u$	P_u in Newtons P_{uf} in Newtons h in mm e_u in mm δ_u in mm w_u in N/mm M_u in N-mm
(A-18)	$P_u = P_{uw} + P_{uf}$	P_u, in Newtons P_{uw} in Newtons P_{uf} in Newtons
(A-19)	$M_u \le \phi\, M_n$	M_u in Newton-mm M_n in Newton-mm
(A-20)	$M_n = \left(A_s f_y + P_u\right)\left(d - \frac{a}{2}\right)$	P_u in Newtons h in mm a in mm d in mm A_s in mm^2 f_y in MPa M_u in N-mm
(A-21)	$a = \frac{(A_s f_y + P_u)}{0.85 f'_{AAC}\ b}$	a in mm P_u in Newtons h in mm b in mm A_s in mm^2 f_y in MPa
(A-22)	$\delta_s \le 0.0007\ h$	δ_s in mm h in mm

Code Eq. No. or Sec. No.	SI Unit Equivalent Equation	Units
(A-23)	$\delta_s = \frac{5M_{cr}h^2}{48E_{AAC}I_g}$	I_g in mm^4 h in mm E_{AAC} in MPa M_{cr} in N-mm
(A-24)	$\delta_s = \frac{5M_{cr}h^2}{48E_{AAC}I_g} + \frac{5(M_{ser} - M_{cr})h^2}{48E_{AAC}I_{cr}}$	I_g in mm^4 h in mm E_{AAC} in MPa M_{cr} in N-mm M_{ser} in N-mm
(A-25)	$M_{cr} = S_n\left(f_{rAAC} + \frac{P}{A_n}\right)$	S_n in mm^3 A_n in mm^2 f_{rAAC} in MPa P in Newtons M_{cr} in N-mm.
(A-26)	$V_{cr} = \frac{S_n}{h}\left(f_{rAAC} + \frac{P}{A_n}\right)$	S_n in mm^3 A_n in mm^2 h in mm f_{rAAC} in MPa P in Newtons V_{cr} in Newtons
A.3.6.8 (a)	$c \geq \frac{l_w}{600\left(C_d\delta_{ne}/h_w\right)}$	c in mm h_w in mm l_w in mm δ_{ne} in mm

Specification for Masonry Structures
(ACI 530.1-~~05~~-xx/ASCE 6-~~05~~-xx/TMS 602-~~05~~-xx)
Reported by the Masonry Standards Joint Committee (MSJC)

PREFACE

P1. This Preface is included for explanatory purposes only; it does not form a part of Specification ACI 530.1-~~05~~-xx/ASCE 6-~~05~~-xx/TMS 602-~~05~~-xx.

P2. Specification ACI 530.1-~~05~~-xx/ASCE 6-~~05~~-xx/TMS 602-~~05~~-xx is a reference standard which the Architect/ Engineer may cite in the contract documents for any project, together with supplementary requirements for the specific project.

P3. Specification ACI 530.1-~~05~~-xx/ASCE 6-~~05~~-xx/TMS 602-~~05~~-xx is written in the three-part section format of the Construction Specifications Institute, as adapted by ACI. The language is generally imperative and terse.

P4. Specification ACI 530.1-~~05~~-xx/ASCE 6-~~05~~-xx/TMS 602-~~05~~-xx is intended to be used in its entirety by reference in the project specifications. Individual sections, articles, or paragraphs should not be copied into the project specifications since taking them out of context may change their meaning.

P5. These mandatory requirements should designate the specific qualities, procedures, materials, and performance criteria for which alternatives are permitted or for which provisions were not made in this Specification. Exceptions to this Specification should be made in the project specifications, if required.

P6. A statement such as the following will serve to make Specification ACI 530.1-~~05~~-xx/ASCE 6-~~05~~-xx/TMS 602-~~05~~-xx an official part of the project specifications:

Masonry construction and materials shall conform to the requirements of "Specification for Masonry Structures (ACI 530.1-~~05~~-xx/ASCE 6-~~05~~-xx/TMS 602-~~05~~-xx)," published by The Masonry Society, Boulder, Colorado; the American Concrete Institute, Farmington Hills, Michigan; and the American Society of Civil Engineers, Reston, Virginia, except as modified by the requirements of these contract documents.

PART 1 — GENERAL

1.1 — Summary

1.1 A. This Specification covers 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. The Specification supplements the legally adopted building code and governs the construction of masonry elements designed in accordance with the Code, except where this Specification is in conflict with requirements in the legally adopted building code. In areas without a legally adopted building code, this Specification defines the minimum acceptable standards of construction practice.

1.1 C. This article covers the furnishing and construction of masonry including the following:

1. Furnishing and placing masonry units, grout, mortar, masonry lintels, sills, copings, through-wall flashing, and connectors.
2. Furnishing, erecting and maintaining of bracing, forming, scaffolding, rigging, and shoring.
3. Furnishing and installing other equipment for constructing masonry.
4. Cleaning masonry and removing surplus material and waste.
5. Installing lintels, nailing blocks, inserts, window and door frames, connectors, and construction items to be built into the masonry, and building in vent pipes, conduits and other items furnished and located by other trades.

1.2 — Definitions

A. *AAC masonry* – masonry made of autoclaved aerated concrete (AAC) units, manufactured without internal reinforcement, and bonded together using thin- or thick-bed mortar.

B. *Acceptable, accepted* — Acceptable to or accepted by the Architect/Engineer.

C. *Architect/Engineer* — The architect, engineer, architectural firm, engineering firm, or architectural and engineering firm, issuing Drawings and Specifications, or administering the Work under Contract Specifications and Project Drawings, or both.

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

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

F. *Autoclaved aerated concrete* -- low-density cementitious product of calcium silicate hydrates.

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

H. *Cleanouts* — Openings that are sized and spaced to allow removal of debris from the bottom of the grout space.

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

J.*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.

K. *Contract Documents* — Documents establishing the required Work, and including in particular, the Project Drawings and Project Specifications.

L. *Contractor* — The person, firm, or corporation with whom the Owner enters into an agreement for construction of the Work.

M. *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. Thickness is given first, followed by height and then length.

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

O. *Glass unit masonry* — Nonload-bearing masonry composed of glass units bonded by mortar.

P. *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.

Q. *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~~can flow around reinforcement and consolidate within masonry under its own weight without consolidation, while retaining its homogeneity~~.

~~P~~R. *Grout lift* — An increment of grout height within a total grout pour. A grout pour consists of one or more grout lifts.

~~Q~~S. *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.

~~R~~T. *Mean daily temperature* — The average daily temperature of temperature extremes predicted by a local weather bureau for the next 24 hours.

~~S~~U. *Minimum daily temperature* — The low temperature forecast by a local weather bureau to occur within the next 24 hours.

~~T~~V. *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.

~~U~~W. *Otherwise required* — Specified differently in requirements supplemental to this Specification.

~~V~~X. *Owner* — The public body or authority, corporation, association, partnership, or individual for whom the Work is provided.

~~W~~Y. *Partition wall* — An interior wall without structural function.

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

~~Y~~AA. *Prestressed masonry* — Masonry in which internal stresses have been introduced to counteract stresses in masonry resulting from applied loads.

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

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

A~~B~~D. *Prestressing tendon* — Steel element such as wire, bar, or strand, or a bundle of such elements, used to impart prestress to masonry.

A~~C~~E. *Project Drawings* — The Drawings that, along with the Project Specifications, complete the descriptive information for constructing the Work required or referred to in the Contract Documents.

A~~D~~F. *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 his agent.

A~~E~~G. *Quality assurance* — The administrative and procedural requirements established by the Contract Documents to assure that constructed masonry is in compliance with the Contract Documents.

A~~F~~H.*Reinforcement* — Nonprestressed steel reinforcement.

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

A~~H~~J. *Slump flow* — The circular spread of plastic self-consolidating grout, which is evaluated in accordance ASTM C 1611/C 1611M.

AK. *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 documents, and upon which the project design is based.

A~~H~~K~~L~~. *Stack bond* — For the purpose of this Specification, stack bond is other than running bond. Usually the placement of masonry units is such that head joints in successive courses are vertically aligned.

AM~~J~~L.*Stone masonry* — Masonry composed of field, quarried, or cast stone units bonded by mortar.

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

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

AN~~K~~M. *Submit, submitted* — Submit, submitted to the Architect/Engineer for review.

A~~L~~N~~O~~.*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.

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

AQ~~N~~P.*Tendon jacking force* — Temporary force exerted by device that introduces tension into prestressing tendons.

AR~~O~~Q. *Unbonded prestressing tendon* — Prestressing tendon that is not bonded to masonry.

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

ATQS. *Visual stability index (VSI)* — An index, defined in ASTM C 1611/C 1611M, that qualitatively indicates the stability of self-consolidating grout

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

ARTV. *Wall, loadbearing* — A wall carrying vertical loads greater than 200 lb per lineal foot (2919 N/m) in addition to its own weight.

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

AYTV. *When required* — Specified in requirements supplemental to this Specification.

AZUX. *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.

BAVY. *Wythe* — Each continuous vertical section of a wall, one masonry unit in thickness.

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

A. ACI 117-90 Standard Specifications for Tolerances for Concrete Construction and Materials (Reapproved 2002)

B. ACI 315-99 Details and Detailing of Concrete Reinforcement

American National Standards Institute

C. ANSI A 137.1-88 Standard Specification for Ceramic Tile

ASTM International

D. ASTM A 36/A 36M-0105 Specification for Carbon Structural Steel

E. ASTM A 82-~~01~~/A 82 M-05a Specification for Steel Wire, Plain, for Concrete Reinforcement

F. ASTM A 123/A 123M-02 Specification for Zinc (Hot-Dip Galvanized) Coating on Iron and Steel Products

G. ASTM A 153/A153M-~~02~~ 05 Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware

H. ASTM A 185-~~01~~/A 185M-06 Specification for Steel Welded Wire Reinforcement, Plain, for Concrete

I. ASTM A 240/A 240M-~~02a~~ 06 Specification for Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications

J. ASTM A 307-~~02~~ 04 Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength

K. ASTM A 416/A 416M-05 ~~02~~ Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete

L. ASTM A 421/A 421M-05 ~~02~~ Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete

M. ASTM A 480/A 480M-~~02~~ 05 Specification for General Requirements for Flat-Rolled Stainless and Heat-Resisting Steel Plate, Sheet, and Strip

N. ASTM A 496-~~01~~/A 496M-05 Specification for Steel Wire, Deformed, for Concrete Reinforcement

O. ASTM A 497/A 497M-06-~~01~~ Specification for Steel Welded Reinforcement, Deformed, for Concrete

P. ASTM A510-~~02~~ 03 General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel

Q. ASTM A 580/A 580M-~~98~~ 06 Specification for Stainless Steel Wire

R. ASTM A 615/A 615M-~~01b~~ 06 Specification for Deformed and Plain ~~Billet~~Carbon-Steel Bars for Concrete Reinforcement

S. ASTM A 641/A 641M-~~98~~ 03 Specification for Zinc-Coated (Galvanized) Carbon Steel Wire

T. ASTM A 653/A 653M-~~02a~~ 05a Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvanealed) by the Hot-Dip Process

U. ASTM A 666-~~00~~ 03 Specification for Annealed or Cold-Worked Austenitic Stainless Steel, Sheet, Strip, Plate and Flat Bar

V. ASTM A 706/A 706M-~~01~~ 06 Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement

W. ASTM A 722/A 722M-00 (2005) ~~98~~ Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete

X. ASTM A 767/ A 767M-~~00b~~05 Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement

Y. ASTM A 775/A 775M-01 Specification for Epoxy-Coated ~~Reinforcing~~ Steel Reinforcing Bars

Z. ASTM A 884/A 884M-~~01~~ 04 Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement

AA. ASTM A 899-91(2002) Specification for Steel Wire Epoxy-Coated

AB. ASTM A 951-~~00~~ 02 Specification for Masonry Joint Reinforcement

AC. ASTM A 996/A 996M-~~01a~~ 06 Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete Reinforcement

AD. ASTM A 1008/ A 1008M-~~02~~~~e1~~05b Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, ~~and~~ High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Baked Hardenable

AE. ASTM B 117-~~02~~ 03 Practice for Operating Salt Spray (Fog) Testing Apparatus

AF. ASTM C 34-~~96 (2001)~~03 Specification for Structural Clay Load-Bearing Wall Tile

AG. ASTM C 55-~~01a~~ 03 Specification for Concrete Brick

AH. ASTM C 56-~~96 (2001)~~05 Specification for Structural Clay Non-Load-Bearing Tile

AI. ASTM C 62-~~01~~ 05 Specification for Building Brick (Solid Masonry Units Made from Clay or Shale)

AJ.ASTM C 67-~~02c~~ 05 Test Methods for Sampling and Testing Brick and Structural Clay Tile

AK. ASTM C 73-~~99a~~ 05 Specification for Calcium Silicate Face Brick (Sand-Lime Brick)

AL. ASTM C 90-~~02~~ 06 Specification for Load-Bearing Concrete Masonry Units

AM. ASTM C 109/C109M-~~02~~ 05 Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens)

AN. ASTM C 126-99 (2005) Specification for Ceramic Glazed Structural Clay Facing Tile, Facing Brick, and Solid Masonry Units

AO. ASTM C 129-~~01~~ 05 Specification for ~~Non-Load-Bearing~~Nonloadbearing Concrete Masonry Units

AP. ASTM C 140-~~02a~~ 05a Test Methods for Sampling and Testing Concrete Masonry Units

AQ. ASTM C 143/C 143-~~00~~ M-05a Test Method for Slump of Hydraulic Cement Concrete

AR. ASTM C 144-~~02~~ 04 Specification for Aggregate for Masonry Mortar

AS. ASTM C150-~~02a~~ 05 Specification for Portland Cement

AT. ASTM C 212-00 Specification for Structural Clay Facing Tile

AU. ASTM C 216-~~02~~ 05a Specification for Facing Brick (Solid Masonry Units Made from Clay or Shale)

AV. ASTM C 270-~~02~~ 05a Specification for Mortar for Unit Masonry

AW. ASTM C 476-02 Specification for Grout for Masonry

AX. ASTM C 482-02 Test Method for Bond Strength of Ceramic Tile to Portland Cement

AY. ASTM C 503-~~99~~$^{\varepsilon 1}$05 Specification for Marble Dimension Stone (Exterior)

AZ. ASTM C 568-~~99~~ 03 Specification for Limestone Dimension Stone

BA. ASTM C 615-~~99~~ 03 Specification for Granite Dimension Stone

BB. ASTM C 616-~~99~~ 03 Specification for Quartz-Based Dimension Stone

BC. ASTM C 629-99 Specification for Slate Dimension Stone

BD. ASTM C 652-~~01a~~ 05a Specification for Hollow Brick (Hollow Masonry Units Made from Clay or Shale)

BE. ASTM C 744-~~99~~ 05 Specification for Prefaced Concrete and Calcium Silicate Masonry Units

BF. ASTM C 901-01 Specification for Prefabricated Masonry Panels

BG. ASTM C 920-~~02~~ 05 Specification for Elastomeric Joint Sealants

BH. ASTM C 1006-84 (2001) Test Method for Splitting Tensile Strength of Masonry Units

BI. ASTM C 1019-~~02~~ 05 Test Method for Sampling and Testing Grout

BJ. ASTM C 1072-~~00a~~ 05b Standard Test Method for Measurement of Masonry Flexural Bond Strength

BK. ASTM C 1088-~~02~~ 05a Specification for Thin Veneer Brick Units Made from Clay or Shale

BL. ASTM C 1314-~~02a~~ 03b Test Method for Compressive Strength of Masonry Prisms

BM. ASTM C 1386-98 Specification for Precast Autoclaved Aerated Concrete (PAAC) Wall Construction Units

BN. ASTM C 1405-05a Specification for Glazed Brick (Single Fired Brick Units)

BO. ASTM C1611/C1611M-05 Standard Test Method for Slump Flow of Self-Consolidating Concrete

BQN. ASTM D 92-~~02b~~ 05a Test Method for Flash and Fire Points by Cleveland Open Cup Tester

BOR. ASTM D 95-~~99~~05$^{\varepsilon 1}$ Test Method for Water in Petroleum Products and Bituminous Material by Distillation

BPS. ASTM D 512-~~89 (1999)~~04 Test Method for Chloride Ion in Water

BQT. ASTM D 566-02 Test Method for Dropping Point of Lubricating Grease

BRU. ASTM D 610-01 Test Method for Evaluating Degree of Rusting on Painted Steel Surfaces

BSV. ASTM D 638-~~02a~~ 03 Test Method for Tensile Properties of Plastics

BTX. ASTM D 994-98 (2003) Specification for Preformed Expansion Joint Filler for Concrete (Bituminous Type)

BUW. ASTM D 1056-00 Specification for Flexible Cellular Materials — Sponge or Expanded Rubber

BVX. ASTM D 1187-97 (2002)$^{\varepsilon 1}$ Specification for Asphalt-Base Emulsions for Use as Protective Coatings for Metal

BWY. ASTM D 1227-95 (2000) Specification for Emulsified Asphalt Used as a Protective Coating for Roofing

BXZ. ASTM D 2000-~~01~~ 05 Classification System for Rubber Products in Automotive Applications

~~BY~~CA. ASTM D 2265-00 Test Method for Dropping Point of Lubricating Grease Over Wide Temperature Range

~~BZ~~CB. ASTM D 2287-96 (2001) Specification for Nonrigid Vinyl Chloride Polymer and Copolymer Molding and Extrusion Compounds

C~~A~~C. ASTM D 4289-~~97~~ 03 Test Method for Elastomer Compatibility of Lubricating Greases and Fluids

C~~B~~D. ASTM E 72-~~02~~ 05 Standard Test Methods of Conducting Strength Tests of Panels for Building Construction

C~~C~~E. ASTM E 328-~~86 (1996)$^{\varepsilon 1}$~~02 Test Methods for Stress Relaxation Tests for Materials and Structures

C~~D~~F. ASTM E 518 ~~E518~~-03 Standard Test Methods for Flexural Bond Strength of Masonry

C~~E~~G. ASTM E519-02 Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages

C~~F~~H. ASTM F 959~~-02~~M-04 Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners [Metric]

American Welding Society

C~~G~~I. AWS D 1.4-05~~98~~ Structural Welding Code – Reinforcing Steel

Federal Test Method Standard

C~~H~~J. FTMS 791B (1974) Oil Separation from Lubricating Grease (Static Technique). 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

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 f'_{mi}, which shall be less than or equal to f'_m.

1.4 B. *Compressive strength determination*

1. *Alternatives 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*
   a. *Clay masonry* — Determine the compressive strength of masonry based on the strength of the units and the type of mortar specified using Table 1. The following Articles must be met:
      1) Units conform to ASTM C 62, ASTM C 216, or ASTM C 652 and are sampled and tested in accordance with ASTM C 67.
      2) Thickness of bed joints does not exceed $^5/_8$ in. (15.9 mm).
      3) For grouted masonry, the grout meets one of the following requirements:
         a) Grout conforms to ASTM C 476.

b) Grout compressive strength equals or exceeds f'_m but compressive strength is not less than 2000 psi (13.79 MPa). Determine compressive strength of grout in accordance with ASTM C 1019.

b. *Concrete masonry* — Determine the compressive strength of masonry based on the strength of the unit and type of mortar specified using Table 2. The following Articles must be met:

1) Units conform to ASTM C 55 or ASTM C 90 and are sampled and tested in accordance with ASTM C 140.

2) Thickness of bed joints does not exceed $^5/_8$ in. (15.9 mm).

3) For grouted masonry, the grout meets one of the following requirements:

a) Grout conforms to ASTM C 476.

b) Grout compressive strength equals or exceeds f'_m but compressive strength is not less than 2000 psi (13.79 MPa). Determine compressive strength of grout in accordance with ASTM C 1019.

c. *AAC masonry* — Determine the compressive strength of masonry based on the strength of the AAC masonry unit only. The following Articles must be met:

1) Units conform to ASTM C 1386.

2) Thickness of bed joints does not exceed 1/8 in. (3.2 mm).

3) For grouted masonry, the grout meets one of the following requirements:

a) Grout conforms to ASTM C 476.

b) Grout compressive strength equals or exceeds f'_{AAC} but compressive strength is not less than 2000 psi (13.79 MPa). Determine compressive strength of grout in accordance with ASTM C 1019.

3. *Prism test method* — Determine the compressive strength of clay masonry and concrete masonry by the prism test method in accordance with ASTM C 1314.

1.4 C. *Adhered veneer requirements* — Determine the adhesion of adhered veneer unit to backing in accordance with ASTM C 482.

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 units, psi (MPa)		Net area compressive strength of masonry, psi (MPa)
Type M or S mortar	Type N mortar	
1700 (11.72)	2100 (14.48)	1000 (6.90)
3350 (23.10)	4150 (28.61)	1500 (10.34)
4950 (34.13)	6200 (42.75)	2000 (13.79)
6600 (45.51)	8250 (56.88)	2500 (17.24)
8250 (56.88)	10,300 (71.02)	3000 (20.69)
9900 (68.26)	—	3500 (24.13)
~~13,200 (91.01)~~11,500 (79.29)	—	4000 (27.58)

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 units, psi (MPa)		Net area compressive strength of masonry, psi[1] (MPa)
Type M or S mortar	Type N mortar	
~~—~~ ~~1250 (8.62)~~	~~1300~~ 1900 (~~8.96~~13.10) 2150 (14.82)	~~1000~~ 1350 (~~6.90~~9.31) 1500 (10.34)
1900 (13.10)	3050 (21.03)	2000 (13.79)
2800 (19.31)	4050 (27.92)	2500 (17.24)
3750 (25.86)	5250 (36.20)	3000 (20.69)
4800 (33.10)		

[1] For units of less than 4 in. (102 mm) height, 85 percent of the values listed.

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 C 270, or

2) Mix designs and mortar tests performed in accordance with the property specification of ASTM C 270.

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 C 476, or

2) Mix designs and grout strength test performed in accordance with ASTM C 476, or

3) Compressive strength tests performed in accordance with ASTM C 1019, and slump flow and visual stability index (VSI) as determined by ASTM C 1611.

2. Material certificates — Material certificates for the following, certifying that each material is in compliance.

a. Reinforcement

b. Anchors, ties, fasteners, and metal accessories

c. Masonry units

d. Mortar, thin-bed mortar for AAC, and grout materials

e. Self-consolidating grout

3. Construction procedures

a. Cold weather construction procedures

b. Hot weather construction procedures

1.6 — Quality assurance

1.6 A. *Testing Agency's services and duties*

1. Sample and test in accordance with Table 3, 4, or 5, as specified for the project.

2. 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.

3. 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 deficiency to the Architect/Engineer, Inspection Agency, and Contractor.

4. ~~The Testing Agency and its representatives are neither authorized to revoke, alter, relax, enlarge, or release any requirement of the Contract Documents, or to accept or approve, or reject or disapprove, any portion of the Work.~~

~~5~~. Unless otherwise required, these services will be paid for by the Owner.

1.6 B. *Inspection Agency's services and duties*

1. Inspect and evaluate in accordance with Table 3, 4, or 5, as specified for the project.
2. 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.
3. Furnish inspection reports to the Architect/Engineer and Contractor.
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 deficiency to the Architect/Engineer and to the Contractor.
5. Submit a final signed report stating whether the Work requiring inspection was, to the best of the Inspection Agency's knowledge, in conformance. Submit the final report to the Architect/Engineer and Contractor.
6. ~~The Inspection Agency is neither authorized to revoke, alter, relax, enlarge, or release any requirement of the Contract Documents, nor authorized to approve or accept, or reject or disapprove, any portion of the Work.~~

~~7.~~ Unless otherwise required, these services will be paid for by the Owner.

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 B or C Quality Assurance (Table 4 or Table 5), construct sample panels of masonry walls.
   a. Use materials and procedures accepted for the Work.
   b. The minimum sample panel size is 4 ft (1.22 m) square.
2. The acceptable standard for the Work is established by the accepted panel.
3. Retain sample panels at the job 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, and grout space geometry do not conform to the requirements of Articles 3.5 C, 3.5 D, and 3.5 E.

Table 3 — Level A Quality Assurance

~~MINIMUM TESTS AND SUBMITTALS~~	~~MINIMUM INSPECTION~~
~~Certificates for materials used in masonry construction indicating compliance with the contract documents~~	~~Verify compliance with the approved submittals~~

MINIMUM TESTS AND SUBMITTALS
Certificates for materials used in masonry construction indicating compliance ~~with the contract documents~~
MINIMUM INSPECTION
Verify compliance with the approved submittals

Table 4 — Level B Quality Assurance

~~MINIMUM TESTS AND SUBMITTALS~~	~~MINIMUM INSPECTION~~
~~Certificates for materials used in masonry construction indicating compliance with the Contract Documents~~ ~~Verification of f'_m or f'_{AAC} in accordance with Article 1.4 B prior to construction, except where specifically exempted by this Code~~	~~As masonry construction begins, verify that the following are in compliance:~~ ~~• proportions of site-prepared mortar~~ ~~• construction of mortar joints~~ ~~• location of reinforcement, connectors, and prestressing tendons and anchorages~~ ~~• prestressing technique~~ ~~Prior to grouting, verify that the following are in compliance:~~ ~~• grout space~~ ~~• grade and size of reinforcement, and prestressing tendons and anchorages~~ ~~• placement of reinforcement, connectors, and prestressing tendons and anchorages~~ ~~• proportions of site-prepared grout and prestressing grout for bonded tendons~~ ~~• construction of mortar joints~~ ~~Verify that the placement of grout and prestressing grout for bonded tendons is in compliance~~ ~~Observe preparation of grout specimens, mortar specimens, and/or prisms~~ ~~Verify compliance with the required inspection provisions of the contract documents and the approved submittals~~

MINIMUM TESTS AND SUBMITTALS		
Certificates for materials used in masonry construction indicating compliance ~~with the contract documents~~		
Verification of Slump flow and VSI as delivered to the site in accordance with Article 1.5.B.1.b.3) for self-consolidating grout		
Verification of f'_m and f'_{AAC} prior to construction, except where specifically exempted by this Code		
MINIMUM INSPECTION		
Inspection Task	**Frequency [(a)]**	
	Continuous	Periodic
1. Verify compliance with ~~the required inspection provisions of the contract documents and~~ the approved submittals		X
2. As masonry construction begins, verify that the following are in compliance:		
a. Proportions of site-prepared mortar		X
b. Construction or mortar joints		X
c. Grade and size of prestressing tendons and anchorages		X
d. Location of reinforcement, connectors, and prestressing tendons and anchorages		X
e. Prestressing technique		X
3. Prior to grouting, verify that the following are in compliance:		
a. Grout space		X
b. Grade, type, and size of reinforcement and anchor bolts, and prestressing tendons, and anchorages		X

c. Placement of reinforcement, connectors, and prestressing tendons and anchorages		X
d. Proportions of site-prepared grout and prestressing grout for bonded tendons		X
e. Construction of mortar joints		X
4. Verify during construction:		
a. Size and location of structural elements		X
b. Type, size, and location of anchors, including other details of anchorage of masonry to structural members, frames, or other construction		X
c. Welding of reinforcement	X	
d. Preparation, construction, and protection of masonry during cold weather (temperature below 40°F (4.4°C)) or hot weather (temperature ~~below~~above 90°F (32.2°C))		X
e. Application and measurement of prestressing force	X	
f. Placement of grout and prestressing grout for bonded tendons is in compliance	X	
5. Observe preparation of grout specimens, mortar specimens, and/or prisms		X

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

Table 5 — Level C Quality Assurance

~~MINIMUM TESTS AND SUBMITTALS~~	~~MINIMUM INSPECTION~~
~~Certificates for materials used in masonry construction indicating compliance with the Contract Documents~~ ~~Verification of f'_m or f'_{AAC} in accordance with Article 1.4 B:~~ ~~• prior to construction~~ ~~• every 5000 sq. ft (464.5 m^2) during construction~~ ~~Verification of proportions of materials in premixed or preblended mortar, grout, and prestressing grout as delivered to the site~~	~~From the beginning of masonry construction and continuously during construction of masonry:~~ ~~a)verify that the following are in compliance:~~ ~~• proportions of site-mixed mortar, grout, and prestressing grout for bonded tendons~~ ~~• grade and size of reinforcement, and prestressing tendons and anchorages~~ ~~• placement of masonry units and construction of mortar joints~~ ~~• placement of reinforcement, connectors, and prestressing tendons and anchorages~~ ~~• grout space prior to grouting~~ ~~• placement of grout and prestressing grout for bonded tendons~~ ~~b)Observe preparation of grout specimens, mortar specimens, and/or prisms~~ ~~c)Verify compliance with the required inspection provisions of the contract documents and the approved submittals~~

MINIMUM TESTS AND SUBMITTALS		
Certificates for materials used in masonry construction indicating compliance ~~with the contract documents~~		
Verification of f'_m and f'_{AAC} in accordance with Article 1.4C prior to construction and for every 5000 sq. ft (464.5 m^2) during construction		
Verification of proportions of materials in premixed or preblended mortar, ~~grout, and~~ prestressing grout, and grout other than self-consolidating grout as delivered to the site		
Verification of Slump flow and VSI as delivered to the site in accordance with Article 1.5.B.1.b.3) for self-consolidating grout		
MINIMUM INSPECTION		
Inspection Task	**Frequency [a]**	
	Continuous	Periodic
1. Verify compliance with ~~the required inspection provisions of the contract documents and~~ the approved submittals		X
2. Verify that the following are in compliance:		
a. Proportions of site-prepared mortar		X
b. Grade, type, and size of reinforcement and anchor bolts, and prestressing tendons and anchorages		X
c. Placement of masonry units and construction of mortar joints		X
d. Placement of reinforcement, connectors, and prestressing tendons and anchorages	X	
e. Grout space prior to grouting	X	
f. Placement of grout and prestressing grout for bonded tendons	X	
g. Size and location of structural elements		X
h. Type, size, and location of anchors including other details of anchorage of masonry to structural members, frames, or other construction	X	
i. Welding of reinforcement	X	
j. Preparation, construction, and protection of masonry during cold weather (temperature below 40°F (4.4°C)) or hot weather (temperature ~~below~~above 90°F (32.2°C))		X
k. Application and measurement of prestressing force	X	

3. Observe preparation of grout specimens, mortar specimens, and/or prisms	X	

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

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.

1.8 — Project conditions

1.8 A. *Construction loads* — Do not apply construction loads that exceed the safe superimposed load-carrying capacity of the masonry and shores, if used.

1.8 B. *Masonry protection* — Cover top of unfinished masonry work to protect it from the weather.

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.
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~~during construction during~~ the following ambient air ~~conditions~~temperatures exist:

a. 40°F to 32°F (4.4°C to 0°C): 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. Grout does not require heated materials, unless the temperature of the materials is below 32°F (0°C).

b. Below 32°F to 25°F (0°C to -3.9°C): 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. 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. 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: Heat masonry surfaces under construction to 40°F (4.4°C) and use wind breaks or enclosures when the wind velocity exceeds 15 mph (24 km/h). Heat masonry to a minimum of 40°F (4.4°C) prior to grouting.

d. Below 20°F (-6.7°C) and below: 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.

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) and below: 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, infared 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.

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).
   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.

PART 2 — PRODUCTS

2.1 — Mortar materials

2.1 A. Provide mortar of the type and color specified, and conforming with ASTM C 270.

2.1 B. *Glass unit masonry*

1. Provide Type S or N mortar that conforms to Article 2.1 A.
2. Comply with the other requirements of Article 2.6.

2.1 C. *AAC Masonry*

1. Provide thin-bed mortar specifically manufactured for use with AAC masonry. Testing to verify mortar properties shall be conducted by the thin-bed mortar manufacturer and confirmed by an independent testing agency.
   a. Provide thin-bed mortar with compressive strength that meets or exceeds the strength of the AAC masonry units. Conduct compressive strength tests in accordance with ASTM C 109.
   b. Provide thin-bed mortar with shear strength that meets or exceeds the strength of the AAC masonry units. Conduct shear strength tests in accordance with ASTM E 519. Cure the gypsum capping for at least 6 hours prior to testing.
   c. For each specified strength class, provide thin-bed mortar with flexural tensile strength that is not less than the smaller of: the maximum value specified in the governing building code; and the modulus of rupture of the masonry units. Conduct flexural strength tests in accordance with ASTM E 72, ASTM E518 Method A or ASTM C 1072.
      1) For conducting flexural strength tests in accordance with ASTM E 518, construct at least five test specimens as stack-bonded prisms at least 32 in. (810 mm) high. Use the type of mortar specified by the AAC unit manufacturer.
      2) For flexural strength tests in accordance with ASTM C 1072, construct test specimens as stack-bonded prisms comprised of at least 3 bed joints. Test a total of at least 5 joints. Use the type of mortar specified by the AAC unit manufacturer.
   d. Perform splitting tensile strength tests in accordance with ASTM C 1006.
2. Mortar for leveling course shall be Type M or S. Conform to the requirements of Article 2.1A.

2.2 — Grout materials

2.2 A. Unless otherwise required, provide grout that conforms to:

1. the requirements of ASTM C 476, or

2. the material requirements of ASTM C476; ~~and~~ attains the specified compressive strength or 2000 psi (13.79 MPa), whichever is greater, at 28 days when tested in accordance with ASTM C1019~~,~~; has a slump flow of 24 in to 30in. (610 to 762 mm) as determined by ASTM C1611; and has a Visual Stability Index (VSI) less than or equal to 1 as determined in accordance with ASTM C1611, Appendix X.1.

2.2 B. Do not use admixtures unless acceptable. Field addition of admixtures is not permitted in self-consolidating grout.

2.3 — Masonry unit materials

2.3 A. Provide concrete masonry units that conform to ASTM C 55, C 73, C 90, C 129, or C 744 as specified.

2.3 B. Provide clay or shale masonry units that conform to ASTM C 34, C 56, C 62, C 126, C 212, C 216, C 652, ~~or~~ C 1088, or C 1405 or to ANSI A 137.1, as specified.

2.3 C. Provide stone masonry units that conform to ASTM C 503, C 568, C 615, C 616, or C 629, 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 C 1386 for the strength class specified in the Contract Documents.

2.4 — Reinforcement, prestressing tendons, and metal accessories

2.4 A. *Reinforcing steel* — Provide deformed reinforcing bars that conform to one of the following as specified:

1.ASTM A 615/A 615M

2.ASTM A 706/A 706M

3.ASTM A 767/A 767M

4.ASTM A 775/A 775M

5.ASTM A 996/A 996M

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 A 421

b. ..... Low-relaxation wire ASTM A 421

c. Strand ..... ASTM A 416/A 416M

d. ..... Low-relaxation strand ASTM A 416/A 416 M

e. Bar ..... ASTM A 722/A 722 M

2. Wire, strands, and bars not specifically listed in ASTM A 416/A416 M, A 421, or A 722/A 722M are permitted, provided they conform to the minimum requirements in ASTM A 416 A/416 M, A 421, or A 722/A 722M and are approved by the Architect/Engineer.

3. Bars and wires of less than 150 ksi (1034 MPa) tensile strength and conforming to ASTM A 82, A 510/A 510M, A 615/A 615M, ASTM A 996/A 996M, or A 706/A 706M are permitted to be used as prestressed tendons, provided that :

a. The stress relaxation properties have been assessed by tests according to ASTM E 328 for the maximum permissible stress in the tendon.

b. Other non-stress-related requirements of Code Chapter 4 addressing prestressing tendons are met.

2.4 C. *Joint reinforcement*

1. Provide joint reinforcement that conforms to ASTM A 951. 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).

2. *Deformed reinforcing wire* — Provide deformed reinforcing wire that conforms to ASTM A 496.

3. *Welded wire fabric* — Provide welded wire fabric that conforms to one of the following specifications:

a. Plain ..... ASTM A 185

b. Deformed ..... ASTM A 497

2.4 D. *Anchors, ties, and accessories* — Provide anchors, ties, and accessories that conform to the following specifications, except as otherwise specified:

1. Plate and bent-bar anchorsASTM A 36/A 36M
2. Sheet-metal anchors and ties.....
   ..... ASTM A 1008/A 1008 M
3. Wire mesh ties.....ASTM A 185
4. Wire ties and anchors .....ASTM A 82
5. Anchor bolts ..... ASTM A 307, Grade A
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 E. *Stainless steel* —Stainless steel items shall be AISI Type 304 or Type 316, and shall conform to the following:

1. .....Joint reinforcement ASTM A 580
2. ..... Plate and bent-bar anchors
   ..... ASTM A 480 and ASTM A 666
3. .....Sheet-metal anchors and ties
   ..... ASTM A 480 and ASTM A 240
4. ..... Wire ties and anchors ASTM A 580

2.4 F. *Coatings for corrosion protection* — Unless otherwise required, protect carbon steel joint reinforcement, ties, and anchors from corrosion by galvanizing or epoxy coating in conformance with the following minimums:

1. Galvanized coatings:
   a. Mill galvanized coatings:
      1) Joint reinforcement .....
         ASTM A 641(0.1 oz/ft^2) (0.031 kg/m^2)
      2) Sheet-metal ties and sheet-metal anchors .....
         ASTM A 653 Coating Designation G60
   b. .....Hot-dip galvanized coatings:
      1) Joint reinforcement, wire ties, and wire anchors
         ASTM A 153 (1.50 oz/ft^2) (458 g/m^2)

2) Sheet-metal ties and sheet-metal anchors .....
.....ASTM A 153 Class B
3) Steel plates and bars (as applicable to size and form indicated) .....
ASTM A 123 or ASTM A 153, Class B

2. Epoxy coatings:
a. Joint reinforcement.....
..... ASTM A 884 Class A
Type 1 —≥ 7 mils (175 μm)
b. ..... Wire ties and anchors .....
ASTM A 899 Class C — 20 mils (508 μm)
c. Sheet-metal ties and anchors .....
..... 20 mils (508 μm) per surface
or manufacturer's specification

2.4 G. *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 (corrosive environment). 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 G.1.a. Fill ducts with prestressing grout complying with Article 2.4 G.1.b.
   a. Ducts — High-density polyethylene or polypropylene.
      1) Use ducts that are mortar-tight and non-reactive with masonry, tendons, and grout.
      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 C 150, Type I, II, or III, that corresponds to the type upon which selection of prestressing grout was based.

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.

~~7) Use sand that conforms to ASTM C 144.~~

2. *Unbonded tendons* — Coat unbonded tendons with a material complying with Article 2.4 G.2b and covered with a sheathing complying with Article 2.4 G.2a. Acceptable materials include a corrosion-inhibiting coating material with a tendon covering (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 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 not less than 0.025 in. (0.6 mm). In corrosive environments, a sheathing thickness of not less than 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.

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 6 for the corrosion-inhibiting coating material.

3. Alternative methods of corrosion protection that provide a protection level equivalent to Articles 2.4 G.1 and 2.4 G.2 are permitted. Stainless steel prestressing tendons or tendons galvanized according to ASTM A 153, 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 H. *Prestressing anchorages, couplers, and end blocks*

1. 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. Place couplers where accepted by Architect/Engineer. Enclose with housing that permits anticipated movements of the couplers during stressing.

3. Protect anchorages, couplers, and end fittings against corrosion.

4. Protect exposed anchorages, couplers, and end fittings to achieve the required mechanical protection and fire rating for the element as specified by local building codes.

Table 6 — Performance specification for corrosion-inhibiting coating

Test	Test Method	Acceptance Criteria
Dropping Point, °F (°C)	ASTM D 566 or ASTM D 2265	Minimum 300 (148.9)
Oil Separation @ 160° F (71.1°C) % by weight	FTMS 791B Method 321.2	Maximum 0.5
Water, % maximum	ASTM D 95	0.1
Flash Point, °F (°C) (Refers to oil component)	ASTM D 92	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 B 117	For normal environments: Rust Grade 7 or better after 720 hr of exposure according to ASTM D 610. For corrosive environments : Rust Grade 7 or better after 1000 hr of exposure according to ASTM D 610.[1]
Water Soluble Ions[2] a. Chlorides, ppm maximum b. Nitrates, ppm maximum c. Sulfides, ppm maximum	ASTM D 512	 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 B 117 (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 D 4289 ASTM D 638	 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 (100 ± 10 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.

2.5 — Accessories

2.5 A. Unless otherwise required, provide contraction (shrinkage) joint material that conforms to one of the following standards:

1. ASTM D 2000, M2AA-805 Rubber shear keys with a minimum durometer hardness of 80.
2. ASTM D 2287, Type PVC 654-4 PVC shear keys with a minimum durometer hardness of 85.
3. ASTM C 920.

2.5 B. Unless otherwise required, provide expansion joint material that conforms to one of the following standards:

1. ASTM C 920.
2. ASTM D 994.
3. ASTM D 1056, Class 2A1.

2.5 C. *Asphalt emulsion* — Provide asphalt emulsion as follows:

1. ..... Metal surfaces ASTM D 1187, Type II
2. Porous surfacesASTM D 1227, 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.

2.6 — Mixing

2.6 A. *Mortar*

1. Mix cementitious materials and aggregates between 3 and 5 min. in a mechanical batch mixer with a sufficient amount of water to produce a workable consistency. Unless acceptable, 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 maximum percentage of mineral oxide or carbon black job-site pigments by weight of cement as follows:

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

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.

2.6 B. *Grout*

1. Unless otherwise required, proportion and mix grout other than self-consolidating grout in accordance with the requirements of ASTM C 476.

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. Job-site proportioning of self-consolidating grout is not permitted. Do not add water at the job site except in accordance with the self-consolidating grout manufacturer's recommendations.

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. *Reinforcement*

1. Fabricate bars used in masonry reinforcement in accordance with the fabricating tolerances of ACI 315.

2. Unless otherwise required, bend bars cold and do not heat bars.

3. The minimum inside diameter of bend for stirrups shall be five bar diameters.
4. Do not bend Grade 40 bars in excess of 180 degrees. The minimum inside diameter of bend is five bar diameters.
5. The minimum inside bend diameter for other bars is as follows:
   a. No. 3 through No. 8 (M#10 through 25) 6 bar diameters
   b. No. 9 through No. 11 (M#29 through 36) 8 bar diameters
6. Provide standard hooks that conform to the following:
   a. A standard 180-degree hook: 180-degree bend plus a minimum extension of 4 bar diameters or $2^1/_2$ in. (64 mm), whichever is greater.
   b. A standard 135-degree hook: a 135-degree bend plus a minimum extension of 6 bar diameters or 4 in. (102 mm), whichever is greater.
   c. A standard 90-degree hook: 90-degree bend plus a minimum extension of 12 bar diameters.
   d. For stirrups: a 90- or 135-degree bend plus a minimum of 6 bar diameters or $2^1/_2$ in. (64 mm), whichever is greater.
7. Fabricate joint reinforcement, anchors, and ties in accordance with this Specification and with the published specifications of the accepted manufacturer.

2.7 B. *Prefabricated masonry*

1. Unless otherwise required, provide prefabricated masonry that conforms to the provisions of ASTM C 901.
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.

PART 3 — EXECUTION

3.1 — Inspection

3.1 A. Prior to the start of masonry construction, the Contractor shall verify:

1. That foundations are constructed with tolerances conforming to the requirements of ACI 117.
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 a combination of both are acceptable without cleaning or brushing provided the dimensions and weights, including heights of deformations, of a cleaned sample are not less than required by the ASTM specification covering this reinforcement in this Specification.

3.2 B. Prior to placing masonry, remove laitance, loose aggregate, and anything else that would prevent mortar from bonding to the foundation.

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 C 67, 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 dropping, debris, loose aggregates, and any material deleterious to masonry grout.

3.2 E. *Reinforcement* — Place reinforcement and ties in grout spaces prior to grouting.

3.2 F. *Cleanouts* — Provide cleanouts in the bottom course of masonry for each grout pour when the grout pour height exceeds 5 ft (1.52 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 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. Construct bed joint of the starting course of foundation with a thickness not less than $^{1}/_{4}$ in. (6.4 mm) and not more than $^{3}/_{4}$ in. (19.1 mm). Provide glass unit masonry bed and head joint thicknesses in accordance with Article 3.3 B.5.c. 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.
2. *Collar joints* — Unless otherwise required, solidly fill collar joints less than $^{3}/_{4}$ in. (19.1 mm) wide with mortar as the job progresses.
3. *Hollow units* — Place hollow units so:
   a. Face shells of bed joints are fully mortared.
   b. Webs are fully mortared in all courses of piers, columns and pilasters, in the starting course on foundations, and when necessary to confine grout or loose-fill 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.
4. *Solid units* — Unless otherwise required, solidly fill bed and head joints with mortar 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. *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 $^1/_4$-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.

6. *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.

7. *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 approximately 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.

3.3 C. *Placing adhered veneer*

1. Brush a paste of neat portland cement on the backing and on the back of the veneer unit.
2. Apply Type S mortar to the backing and to the veneer unit.
3. Tap the veneer unit into place, completely filling the space between the veneer unit and the backing. Sufficient mortar shall be used to create a slight excess to be forced out between the edges of the veneer units. The resulting thickness of the mortar in back of the veneer unit shall not be less than $^3/_8$ in. (9.5 mm) nor more than 1¼ in. (31.8 mm).
4. Tool the mortar joint with a round jointer when the mortar is thumbprint hard.

3.3 D. *Prefabricated concrete and masonry items* — Erect prefabricated concrete and masonry items in accordance with the requirements.

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 nonbearing masonry partitions.
3. Place pipes and conduits passing horizontally through piers, 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.
6. Install movement joints.
7. Aluminum — Do not embed aluminum conduits, pipes, and accessories in masonry, grout, or mortar, unless effectively coated or covered 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. Dimension of elements

a. In cross section or elevation

.....-$^1/_4$ in. (6.4 mm), +$^1/_2$ in. (12.7 mm)

b. ..... Mortar joint thickness

bed .....±$^1/_8$ 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.5.c

c. Grout space or cavity width, except for masonry walls passing framed construction

.....-$^1/_4$ in. (6.4 mm), + $^3/_8$ in. (9.5 mm)

2. Elements

a. Variation from level:

bed joints

.....±$^1/_4$ in. (6.4 mm) in 10 ft (3.05 m)

.....±$^1/_2$ in. (12.7 mm) maximum

top surface of bearing walls

.....±$^1/_4$ in. (6.4 mm) in 10 ft (3.05 m)

.....±$^1/_2$ in. (12.7 mm) maximum

b. Variation from plumb

.....±$^1/_4$ in. (6.4 mm) in 10 ft (3.05 m)

.....±$^3/_8$ in. (9.5 mm) in 20 ft (6.10 m)

.....±$^1/_2$ in. (13 mm) maximum

c. True to a line

.....±$^1/_4$ in. (6.4 mm) in 10 ft (3.05 m)

.....±$^3/_8$ in. (9.5 mm) in 20 ft (6.10 m)

.....±$^1/_2$ in. (12.7 mm) maximum

d. Alignment of columns and walls

(bottom versus top)

±$^1/_2$ in. (12.7 mm) for bearing walls and columns

±$^3/_4$ in. (19.1 mm) for nonbearing walls

3. Location of elements

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.

3.4 — Reinforcement, tie, and anchor installation

3.4 A. *Basic requirements* — Place reinforcement, wall 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 and fasten reinforcement together to prevent displacement beyond the tolerances allowed by construction loads or by placement of grout or mortar.
2. Completely embed reinforcing bars in grout in accordance with Article 3.5.
3. Maintain clear distance between reinforcing bars and any face of masonry unit or formed surface, but not less than $^1/_4$ in. (6.4 mm) for fine grout or $^1/_2$ in. (12.7 mm) for coarse grout.
4. Splice only where indicated on the Project Drawings, unless otherwise acceptable. When splicing by welding, provide welds in conformance with the provisions of AWS D 1.4.
5. Unless accepted by the Architect/Engineer, do not bend reinforcement after it is embedded in grout or mortar.
6. *Noncontact lap splices* — Position bars spliced by non-contact lap splice no farther apart transversely than one-fifth the specified length of lap nor more than 8 in. (203 mm)
~~6~~7. *Joint reinforcement*
   a. Place joint reinforcement 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 and $^5/_8$ in. (15.9 mm) when exposed to weather or earth.
   a. Provide minimum 6 in. (152.4 mm) lap splices for joint reinforcement.

b. Ensure that all ends of longitudinal wires of joint reinforcement are embedded in mortar at laps.

~~7~~8. *Placement tolerances*

a. Tolerances for the placement of steel in walls and flexural elements shall be ± $^1/_2$ in. (12.7 mm) when the distance from the centerline of steel 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 within 2 in. (50.8 mm) of the required location along the length of the wall.

c. If it is necessary to move bars more than one bar 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 bars.

d. Foundation dowels that interfere with unit webs are permitted to be bent ~~up~~ to a maximum of 1 in. (25 mm) horizontally for every 6 in. (152 mm) of vertical height.

3.4 C. *Wall ties*

1. Embed the ends of wall ties in mortar joints. Embed wall tie ends at least $^1/_2$ in. (13 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.

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. The maximum misalignment of bed joints from one wythe to the other is $1^1/_4$ in. (31.8 mm).
   d. The maximum clearance between connecting parts of the ties is $^1/_{16}$ in. (1.6 mm).
   e. When pintle legs are used, provide ties with at least two legs made of wire size W2.8 (MW18).
5. Install wire ties perpendicular to a vertical line on the face of the wythe from which they protrude. Where one-piece ties or joint reinforcement are used, the bed joints of adjacent wythes shall align.
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.

3.4 D. *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 may be placed in grout or mortar bed joints that have a specified thickness of at least ½ in. (12.7 mm) thickness.
2. Maintain clear distance between anchor bolts and any face of masonry unit or formed surface of at least ¼ in. (6.4 mm) when using fine grout, and of at least ½ in. (12.7 mm) when using coarse grout.
3. 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).

3.4 ~~D~~E. *Veneer anchors* — Place corrugated sheet-metal anchors, sheet-metal anchors, and wire anchors as follows:

1. With solid units, embed anchors in mortar joint and extend into the veneer a minimum of 1 ½ in. (38.1 mm), with at least $^5/_8$ in. (15.9 mm) cover to the outside face.

2. 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.
3. Install adjustable anchors in accordance with the requirements of Articles 3.4 C.4.c, d, and e.
4. Provide at least one adjustable two-piece anchor, anchor of wire size W 1.7 (MW11), or 22 gage (0.8 mm) corrugated sheet-metal anchor for each 2.67 ft^2 (0.25 m^2) of wall area.
5. Provide at least one anchor of ~~all~~ other types for each 3.5 ft^2 (0.33 m^2) of wall area.
6. Space anchors at a maximum of 32 in. (813 mm) horizontally and ~~18~~ 25 in. (~~457~~635 mm) vertically, but not to exceed the applicable requirement of Article 3.4 D. 4 or 3.4 D. 5.
7. Provide additional anchors around openings larger than 16 in. (406 mm) in either dimension. Space anchors around the perimeter of opening at a maximum of 3 ft (0.9 m) on center. Place anchors within 12 in. (305 mm) of opening.

3.4 ~~E~~F. *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.

3.5 — 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 field-mixed grout that does not meet the specified slump without adding water after initial mixing.
2. For transit-mixed grout:
   a. Addition of water is permitted at the time of initial discharge to adjust slump to conform to Article 2.6 B.2.
   b. Discard transmit-mixed grout that does not meet the specified slump without adding water, other than the water that was added at the time of initial discharge.

c. The time limitation is waived as long as the transit-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.

Table 7 — Grout space requirements

Grout type[1]	Maximum grout pour height, ft (m)	Minimum width of grout space,[2,3] in. (mm)	Minimum grout space dimensions for grouting cells of hollow units,[3,4,5] 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 (1.52)	2 (50.8)	2 x 3 (50.8 x 76.2)
Fine	12 (3.66)	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 (1.52)	2 (50.8)	2 1/2 x 3 (63.5 x 76.2)
Coarse	12 (3.66)	2 1/2 (63.5)	3 x 3 (76.2 x 76.2)
Coarse	24 (7.32)	3 (76.2)	3 x 4 (76 .2x 102)

[1] Fine and coarse grouts are defined in ASTM C 476.

[2] For grouting between masonry wythes.

[3] Grout space dimension is the clear dimension between any masonry protrusion and shall be increased by the diameters of the horizontal bars within the cross section of the grout space.

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

[5] Minimum grout space dimension for AAC masonry units shall be 3-in. x 3-in. or a 3-in. diameter cell.

3.5 D. *Grout lift height*—

1. Where the following conditions are met, place grout in lifts not exceeding 12.67 ft (3.86 m).

   a~~1~~. The masonry has cured for at least 4 hours.

   ~~2~~b. The grout slump is maintained between 10 and 11 in. (254 and 279 mm).

   ~~3~~c. No intermediate reinforced bond beams are placed between the top and the bottom of the pour height.

2. For self-consolidating grout conforming to Article 2.2 ~~where the~~placed in masonry that has cured for at least 4 hours, place in lifts not exceeding the grout pour height.

3. Otherwise, place grout in lifts not exceeding 5 ft (1.52 m).

3.5 E. *Consolidation —*

1. Consolidate grout at the time of placement.

~~1~~a. Consolidate grout pours 12 in. (305 mm) or less in height by mechanical vibration or by puddling.

b~~2~~. 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.

2. Consolidation or reconsolidation is not required for self-consolidating grout.

3.5 F. *Grout key* — When grouting, form grout keys between grout pours.

1. Form a grout key by terminating the grout a minimum of 1½ in. (38.1 mm) below a mortar joint at vertically grouted spaces.
2. Form a grout key between grout lifts when the first lift is permitted to set prior to placement of the subsequent lift.
3. At beams, terminate the grout pour in the course above or the course below the beam and form a grout key. Completely fill the beam except form a grout key at vertically reinforced cells.

3.5 ~~F~~G. *Alternate grout placement* — Place masonry units and grout using construction procedures employed in the accepted grout demonstration panel.

3.5 ~~G~~H. *Grout for AAC masonry* -- Use grout conforming to ASTM C 476. Wet AAC masonry thoroughly before grouting to ensure that the grout flows to completely fill the space to be grouted. Grout slump shall be between 8 in. and 11 in. (203 and 279 mm) when determined in accordance with ASTM C 143.

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 beams, columns, pilasters, and 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 F 959 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 elements or 7 percent for post-tensioned elements, 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.

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.

3.7 — 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 or grout droppings, and debris.

FOREWORD TO SPECIFICATION CHECKLISTS

F1. This Foreword is included for explanatory purposes only; it does not form a part of Specification ACI 530.1-08/ASCE 6-08/TMS 602-08.

F2. Specification ACI 530.1-08/ASCE 6-08/TMS 602-08 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 Specification ACI 530.1-08/ASCE 6-08/TMS 602-08. Checklists assist the Architect/Engineer in selecting and specifying project requirements in the Project Specification.

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 Specification ACI 530.1-08/ASCE 6-08/TMS 602-08 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. The checklists identify the Sections, Parts, and Articles of the reference Specification and the action required or available to the Architect/Engineer.

~~**F1.** Checklists that follow do not form a part of Specification ACI 530.1-05-xx/ASCE 6-05-xx/TMS 602-05-xx. The purpose of these Checklists is to assist the Architect/Engineer in properly choosing and specifying the necessary mandatory and optional requirements for the project specifications.~~

F2. Building codes (of which this standard is a part by reference) set minimum requirements necessary to protect the public. Project specifications often 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 that follow 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 sections, parts, and articles of the Specification, and the second column contains notes to the Architect/Engineer to indicate the type of action required by the Architect/Engineer.

1

MANDATORY REQUIREMENTS CHECKLIST

Section/Part/Article		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 empirically designed masonry. Specify f'_{mi} for prestressed masonry.
1.4 B.2	Unit strength method	Specify when strength of grout is to be determined by test.
1.6	Quality assurance	Define the submittal reporting and review procedure.
1.6 A.1	Testing Agency's services and duties	Specify which of Tables 3, 4, or 5 applies to the project.
1.6 B.1	Inspection Agency's services and duties	Specify which of Tables 3, 4, or 5 applies to the project.
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 C.3	Welded wire fabric	Specify when welded wire fabric is to be plain.
2.4 E	Stainless steel	Specify when stainless steel joint reinforcement, anchors, ties, and/or accessories are required.
2.4 F	Coating for corrosion protection	Specify ~~which interior walls are governed by this provision~~the types of corrosion protection that are required for each portion of the masonry construction.
2.4 G	Corrosion protection for tendons	Specify the corrosion protection method.
2.4 H	Prestressing anchorages, couplers, and end blocks	Specify the anchorages and couplers and their corrosion protection.
2.5 E	Joint fillers	Specify size and shape of joint fillers.
2.7 B	Prefabricated masonry	Specify prefabricated masonry and requirements in supplement of those of ASTM C 901.
PART 3 — EXECUTION		
3.3 E.2	Pipes and conduits	Specify sleeve sizes and spacing.
3.3 E.6	Accessories	Specify accessories not indicated on the project drawings.
3.3 E.7	Movement joints	Indicate type and location of movement joints on the project drawings.

1

OPTIONAL REQUIREMENTS CHECKLIST

Section/Part/Article	Notes to the Architect/Engineer
PART 1 — GENERAL	
1.5 B	Specify required submittals.
1.6 Quality assurance	Define who will pay for testing and inspection services, if other than the Owner.
PART 2 — PRODUCTS	
2.2	Specify grout requirements at variance with ACI 530.1/ASCE 6/TMS 602. Specify admixtures.
2.5 A Movement joint and 2.5 B	Specify requirements at variance with ACI 530.1/ASCE 6/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.
2.6 B.1 Grout proportioning and mixing	Specify requirements at variance with ACI 530.1/ASCE 6/TMS 602
PART 3 — EXECUTION	
3.2 C Wetting masonry units	Specify when units are to be wetted.
3.3 A Bond pattern	Specify bond pattern other than running bond.
3.3 B.1 Bed and head joints	Specify thickness and tooling differing from ACI 530.1/ASCE 6/TMS 602.
3.3 B.2 Collar joints	Specify the filling of collar joints less than $^3/_4$ in. (19.1 mm) thick differing from ACI 530.1/ASCE 6/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 ACI 530.1/ASCE 6/TMS 602.
3.3 B.5 Glass units	Specify mortar bedding at variance with ACI 530.1/ASCE 6/TMS 602.
3.3 B.7.b AAC Masonry	Specify when mortar may be omitted from head joints.
3.3 E.2 Embedded items and accessories	Specify locations where sleeves are required for pipes or conduits.
3.4 C.2, 3, and 4	Specify requirements at variance with ACI 530.1/ASCE 6/TMS 602.

2

Commentary on Building Code Requirements for Masonry Structures (ACI 530-05-xx/ASCE 5-05-xx/TMS 402-05-xx) Reported by the Masonry Standards Joint Committee (MSJC)

INTRODUCTION

This Commentary documents some of the considerations of the Masonry Standards Joint Committee (MSJC) in developing the provisions contained in *Building Code Requirements for Masonry Structures* (ACI 530-05-xx/ASCE 5-05-xx/TMS 402-05-xx), referred to here as the Code. Comments on specific provisions are made under the corresponding chapter and section numbers of this Code.

The Commentary is not intended to provide a detailed account of the studies and research data reviewed by the committee in formulating the provisions of this Code. However, references to some of the research data are provided for those who wish to study the background material in depth.

As the name implies, *Building Code Requirements for Masonry Structures* (ACI 530-05-xx/ASCE 5-05-xx/TMS 402-05-xx) is meant to be used as part of a legally adopted building code and as such must differ in form and substance from documents that provide detailed specifications, recommended practices, complete design procedures, or design aids.

This Code is intended to cover all buildings of the usual types, both large and small. This Code and this Commentary cannot replace sound engineering knowledge, experience, and judgment. Requirements more stringent than the Code provisions may sometimes be desirable.

A building code states only the minimum requirements necessary to provide for public health and safety. The MSJC Building Code is based on this principle. For any structure, the owner or the structural designer may require the quality of materials and construction to be higher than the minimum requirements necessary to protect the public as stated in this Code. However, lower standards are not permitted.

This Commentary directs attention to other documents that provide suggestions for carrying out the requirements and intent of this Code. However, those documents and this Commentary are not intended to be a part of this Code.

This Code has no legal status unless it is adopted by government bodies having the police power to regulate building design and construction or unless incorporated into a contract. Where this Code has not been adopted, it may serve as a reference to good practice even though it has no legal status.

This Code provides a means of establishing minimum standards for acceptance of designs and construction by a legally appointed building official or designated representatives. Therefore, this Code cannot define the contract responsibility of each of the parties in usual construction, unless incorporated into a contract. However, general references requiring compliance with this Code in the project specifications are improper since minimum code requirements should be incorporated in the contract documents, which should contain ~~all~~ requirements necessary for construction.

Masonry is one of the oldest forms of construction. In modern times, the design of masonry has been governed by standards that separate clay masonry from concrete masonry. For this Code, the committee has adopted the policy that the design methodology for all masonry should be the same. The committee adopted this policy in recognition that the design methodology developed does not always predict the actual performance of masonry as accurately as it would like and that masonry work designed in accordance with some empirical provisions performs better than would be indicated by current design procedures. These design situations are being identified by the committee and singled out for further detailed research.

CHAPTER 1
GENERAL DESIGN REQUIREMENTS FOR MASONRY

1.1 — Scope

This Code covers the structural design and construction of masonry elements and serves as a part of the legally adopted building code. Since the requirements for masonry in this Code are interrelated, this Code may need to supersede when there are conflicts on masonry design and construction with the legally adopted building code or with documents referenced by this Code. The designer must resolve the conflict for each specific case.

1.1.21 *Minimum requirements*

This code governs structural design of both structural and non-structural masonry elements. Examples of non-structural elements are masonry veneer, glass unit masonry, and masonry partitions. Structural design aspects of non-structural masonry elements include, but are not limited to, gravity and lateral support, and load transfer to supporting elements.

1.1.3 *Design procedures*

The design procedures in Chapter 2 are allowable stress methods in which the stresses resulting from service loads ~~do~~must not exceed permissible service load stresses. The design procedures in Chapter 3 are strength design methods in which internal forces resulting from application of factored loads must not exceed design strength (nominal member capacity reduced by a strength-reduction factor ϕ).

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.

Service load is the load that is assumed by the legally adopted building code to actually occur when the structure is in service. The stresses allowed under the action of service loads are limited to values within the elastic range of the materials.

For strength design methods, internal forces arising from application of combinations of factored loads are the basis for design. Such load combinations are specified in the legally adopted building

code. Nominal member strengths are typically computed using minimum specified material strengths. Materials are assumed to be homogenous, isotropic, and exhibit nonlinear behavior. Under loads that exceed service levels, nonlinear material behavior, cracking, and reinforcing bar slip invalidate the assumption regarding the linearity of the stress-strain relation for masonry, grout, and reinforcing steel. ~~However, if~~If nonlinear behavior is modeled, however, ~~then~~ capacity can be accurately predicted~~ions of capacity can be achieved~~. 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.

Empirical design procedures of Chapter 5 are permitted in certain instances. ~~Members~~ Elements not working integrally with the structure, such as partition or panel walls, or any ~~member~~ element not (or not permanently) absorbing or transmitting forces resulting from the behavior of the structure under loads, may be designed empirically. A masonry shear wall would be an integral structural ~~part~~ element while some wall partitions, because of their method of construction or attachment, would not. Empirical design is permitted for buildings of limited height and low seismic exposure.

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 7-~~02~~. 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 partitions; adequate continuity of walls, ties, and joint rigidity; providing walls capable of beam action; ductile detailing and the use of compartmentalized construction.

1.2 — Contract documents and calculations

1.2.1 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 ~~thereto~~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 regarding design (Code)

requirements, unless the construction entity is acting in a design-build capacity. A Commentary on ACI 530.1/ASCE 6/TMS 602 follows the Specification.

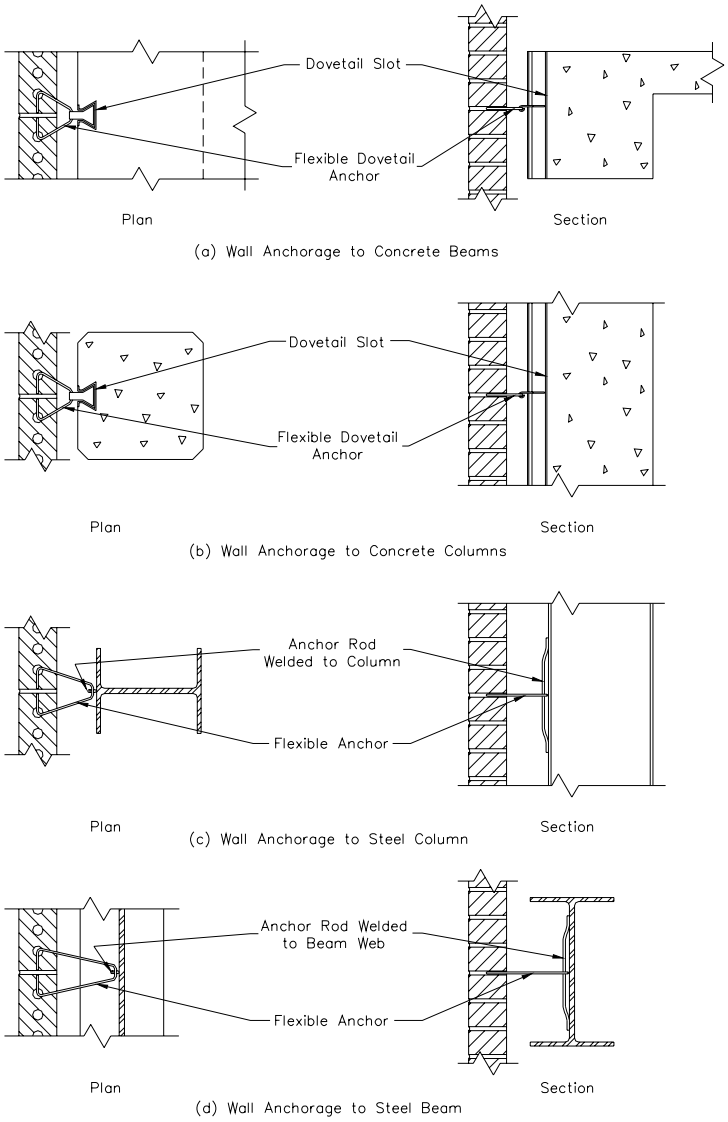
1.2.2 This Code lists some of the more important items of information that must be included in the project drawings or project specifications. This is not an all-inclusive list, and additional items may be required by the building official.

Masonry does not always behave in the same manner as its structural supports or adjacent construction. 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 some connections transfer loads perpendicular to the wall, other devices transfer loads within the plane of the wall. ~~Details shown in~~ Figure~~.~~ 1.2.2-1 ~~are~~shows representative wall anchorage details that~~examples and~~ allow movement within the plane of the wall. While load transfer usually involves masonry attached to structural elements, such as beams or columns, the connection of nonstructural elements, such as door and window frames, should also be ~~investigated~~addressed.

Connectors are of a variety of sizes, shapes, and uses. In order to perform properly they should be identified on the project drawings.

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.

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.



~~Fig.~~Figure 1.2.2-1 — Wall anchorage details

1.3 — Approval of special systems of design or construction

New methods of design, new materials, and new uses of materials must undergo a period of development before being specifically covered in 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 a Board of Examiners. Such a board should be created and named in accordance with local laws and should be headed by a registered engineer. ~~All b~~Board members should be directly associated with, and competent in, the fields of structural design or construction of masonry.

For special systems considered under this section, specific tests, load factors, deflection limits, and other pertinent requirements should be set by the board of examiners, and should be consistent with the intent of the Code.

1.4—Standards cited in this Code

These standards are referenced in this Code. Specific dates are listed here since changes to the standard may result in changes of properties or procedures. ~~Two editions of ASCE 7 are referenced, since some of the provisions in this standard are still based on the earlier edition of ASCE 7. Accordingly, the Architect/Engineer is cautioned to read the provisions carefully to ensure that the appropriate provisions are applied.~~

Contact information for these organizations is given below:

American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
www.aci-int.org

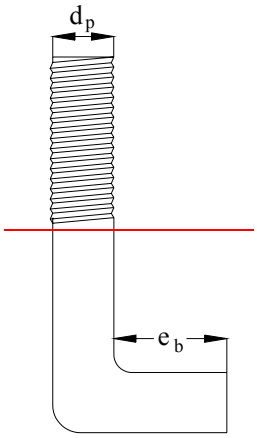
American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, VA 20191
www.asce.org

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

American Welding Society
550 N.W. LeJeune Road
Miami, Florida 33126
www.aws.org

1.5 — Notation

Notations used in this Code are summarized here. ~~Each symbol is unique, and is consistent with with the notation as used in other masonry standards when possible. Figure 1.5-1 graphically shows e_b for a bent-bar anchor bolt.~~



~~*Fig. 1.5-1—Bent-bar anchor bolt*~~

1.6 — 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. Glossaries of masonry terminology are available from several sources within the industry[1.1, 1.2, 1.3].

The permitted tolerances for units are given in the appropriate materials standards. Permitted tolerances for joints and masonry construction are given in the Specification. Nominal dimensions are usually used to identify the size of a masonry unit. The thickness or width is given first, followed by height and length. Nominal dimensions are normally given in whole numbers nearest to the specified dimensions. Specified dimensions are most often used for design calculations.

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

1.7 — Loading

The provisions establish design load requirements. If the ~~service~~ design loads specified by the legally adopted building code differ from those of ASCE 7~~-02~~, the legally adopted building code governs. The Architect/Engineer may decide to use the more stringent requirements.

1.7.3 *Lateral load resistance*

Lateral load resistance must be provided by a braced structural system. Partitions, infill panels, and similar elements may not be a part of the lateral-force-resisting system if isolated. However, when they resist lateral forces due to their rigidity, they should be considered in analysis.

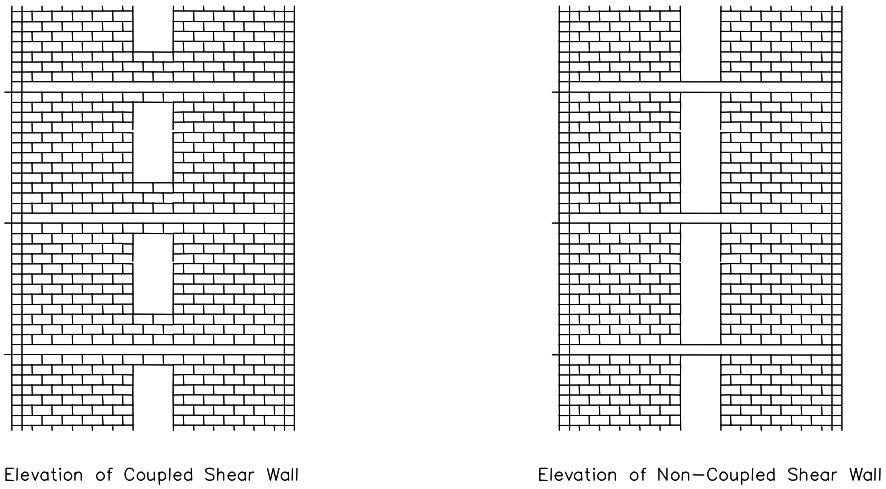
1.7.4 *Other effects*

Service loads are not the sole source of stresses. The structure must 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.

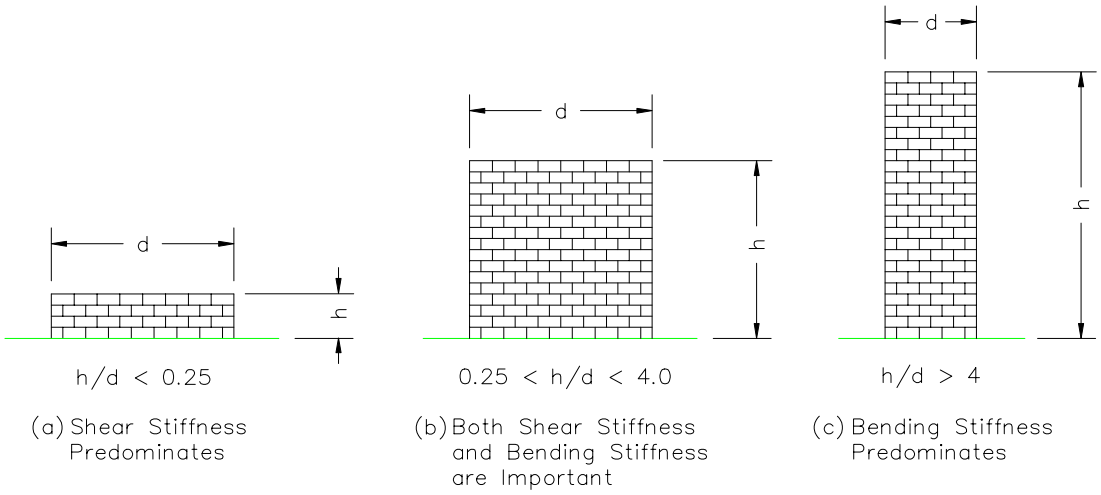
1.7.5 *Lateral load distribution*

The design assumptions for masonry buildings include the use of a ~~braced structural~~lateral load-resisting system. The distribution of lateral loads to the members of the resisting structural system is a function of the rigidities of the structural system and of the horizontal diaphragms. The method of connection at intersecting walls and between walls and floor and roof diaphragms determines if the wall participates in the resisting structural 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. The analysis should rationally consider the effects of openings in shear walls and whether the masonry above the openings allows them to act as coupled shear walls. See ~~Fig.~~Figure 1.7-1. The interaction of coupled shear walls is complex and further information may be obtained from Reference 1.4.



~~Fig.~~Figure 1.7-1 — Coupled and noncoupled shear walls



~~Fig.~~Figure 1.7-2 — Shear wall stiffness

Computation 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 ~~Fig.~~Figure 1.7-2. For nongrouted hollow unit shear walls, the use of equivalent solid thickness of wall in computing web stiffness is acceptable.

1.8 — Material properties

1.8.1 *General*

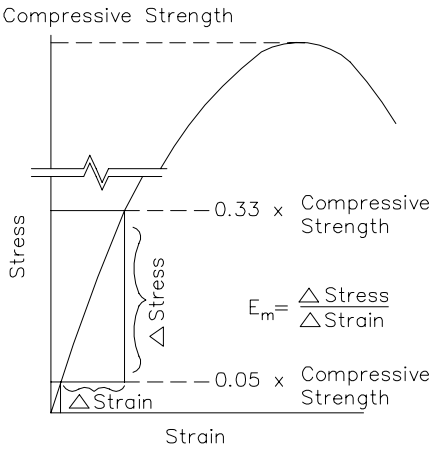
Proper evaluation of the building material movement from all sources is an important element of masonry design. ~~Brick~~ Clay masonry and concrete masonry may behave quite differently under normal loading and weather conditions. The committee has extensively studied available research information in the development of these material properties. However, the Committee recognizes the need for further research on this subject. 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.

1.8.2 *Elastic moduli*

Modulus of elasticity for clay and concrete masonry has traditionally been taken as $1000 f'_m$ in previous masonry codes. Research[1.5, 1.6] has indicated, however, that there is a large variation in the relationship of elastic modulus versus compressive strength of masonry, and that lower values may be more typical. However, variation in procedures between one research investigation and another may account for much of the indicated variation. Furthermore, the type of elastic moduli being reported (that is, secant modulus, tangent modulus, chord modulus, etc.) is not always identified. The committee decided the most appropriate elastic modulus for working-stress design purposes is the slope of the stress-strain curve below a stress value of $0.33 f'_m$, the allowable flexural compressive stress. 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 ~~Fig.~~Figure 1.8-1). 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.

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.[1.5] and results of a test program by Colville et al[1.6]. Code values for E_m are higher than indicated by a best fit of data relating E_m 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.

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 ~~all~~ each part~~s~~ of a composite wall were based on one specified compressive strength of masonry.



~~Fig.~~*Figure 1.8-1 — Chord modulus of elasticity*

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 References A~~8~~.3 and A~~8~~.4.

The modulus of elasticity of a grouted assemblage of clay or concrete masonry can usually be taken as a factor ~~times~~ 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. ~~The grout is usually much stiffer than the AAC material.~~ While it is permissible and conservative to compute 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 element 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 computing element resistances. Research is ongoing to resolve this issue.

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.

1.8.3 *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[1.7, 1.8, 1.9].

Thermal expansion for concrete masonry varies with aggregate type[1.7, 1.10].

Thermal expansion coefficients are given for AAC masonry in Reference [1.~~7~~8~~11~~].

1.8.4 *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[1.8, 1.9]. 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[1.12~~10~~]. ~~Moisture expansion is reversible in concrete masonry.~~ Moisture expansion is not a design consideration for concrete masonry.

1.8.5 *Coefficient of shrinkage*

1.8.5.1 ~~*for Cc*~~*oncrete masonry* — Concrete masonry is a cement-based material that shrinks due to moisture loss and carbonation[1.~~10~~11~~10~~]. The total linear drying shrinkage is determined in accordance with ASTM C 426. The maximum shrinkage allowed by ASTM specifications for concrete masonry units (for example, ASTM C 90), other than calcium silicate units, is 0.065%. Further design guidance for estimating the shrinkage due to moisture loss and carbonation is available[1.13~~18~~19~~20~~, 1.~~19~~20~~21~~14, 1.~~20~~21~~22~~15]. The shrinkage of clay masonry is negligible.

1.8.5.2 *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 C 1386 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.

1.8.6 *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[1.10 10 11, 1.16 11 12, 1.17 8 19 11]. The values given are maximum values.

1.8.7 *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[1.12 13 17]. 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 by the manufacturer. 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).

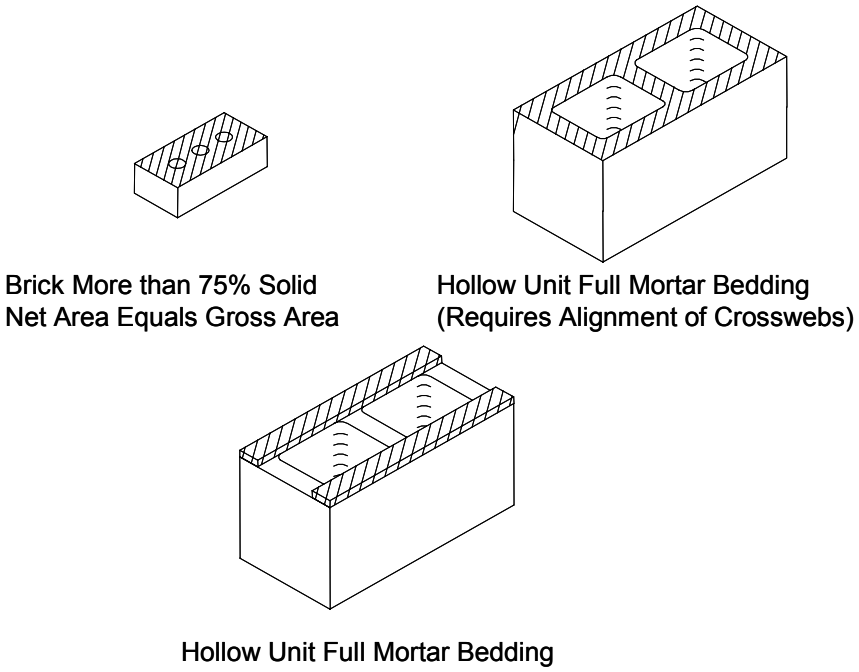
1.9 — Section properties

1.9.1 *Stress computations*

Minimum net section is often difficult to establish in hollow unit masonry. The designer may choose to use the minimum thickness of the face shells of the units as the minimum net section. The minimum net section may not be the same in the vertical and horizontal directions.

For masonry of hollow units, the minimum cross-sectional area in both directions may conservatively be based on the minimum face-shell thickness[1.18 13 14].

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 2.1.5.2.2(c) for masonry headers. Several conditions of net area are shown in ~~Fig.~~ Figure 1.9-1.



~~Fig.~~ Figure 1.9-1 — Net cross-sectional areas

Since 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 compute these stresses, a convenient 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 moduli of elasticity of the ~~weakest~~ most deformable material in the member and 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 computed 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.

1.9.2 *Stiffness*

Stiffness is a function of the extent of cracking. The Code equations for design in Section 2.2, however, are based on the member's uncracked moment of inertia. Also, since the extent of tension cracking in shear walls is not known in advance, this Code allows the determination of stiffness to be based on uncracked section properties. For reinforced masonry, the stiffness calculations based on the cracked section yield more accurate results.

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 computations, an average value of the appropriate section property (~~, that is,~~ 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.

1.9.3 *Radius of gyration*

The radius of gyration is the square root of the ratio of bending moment of inertia to cross-sectional area. Since stiffness is based on the average net cross-sectional area of the member considered, this same area should be used in the computation of radius of gyration.

1.9.4 *Intersecting walls*

Connections of webs to flanges of ~~shear~~ 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. A running bond connection should be as shown in ~~Fig.~~Figure 1.9-2 with a "T" geometry over their intersection.

The alternate method, using~~making use of~~ metal strap connectors, is shown in ~~Fig.~~Figure 1.9-3. Bond beams, shown in ~~Fig.~~Figure 1.9-4, are the third means of connecting webs to flanges.

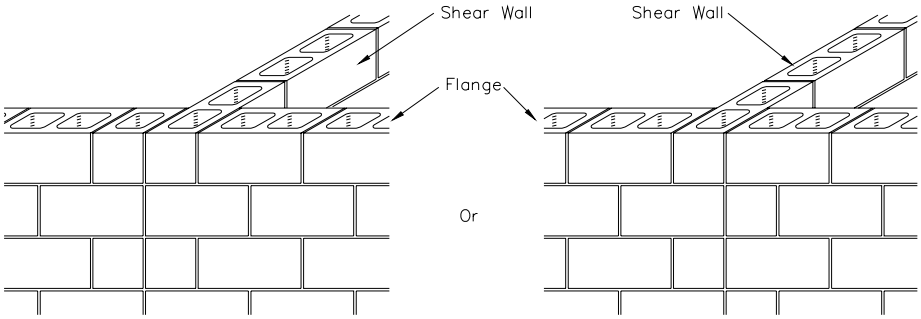
When the flanges are connected at the intersection, they are required to be included in the design. ~~The effective width of the flange is a traditional requirement. The effective flange width is shown in Fig. 1.9-5.~~

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[1.19]. They showed that the shear-lag effects are significant for uncracked walls,

but become less severe after cracking. He and Priestley[1.14] proposed that the effective width of the flange be determined as:

$$l_e = \begin{cases} l & l/h \le 1.5 \\ 0.75h + 0.5l & 1.5 \le l/h \le 3.5 \\ 2.5h & l/h \ge 3.5 \end{cases}$$

where l_e is the effective flange width, l 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 since the testing by He and Priestley[1.19] 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 carrying the load can lead to other, more brittle, failure modes such as shear or crushing of the compression toe.



~~Fig.~~ *Figure 1.9-2 — Running bond lap at intersection*

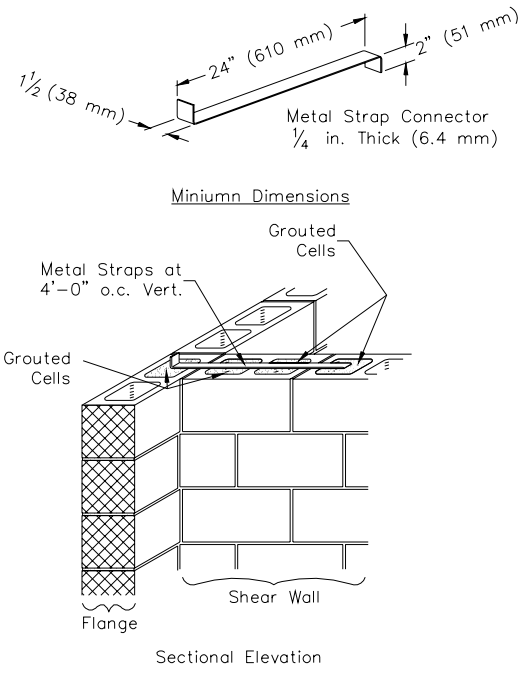


Fig 1.9-3 — Metal straps and grouting at wall intersections

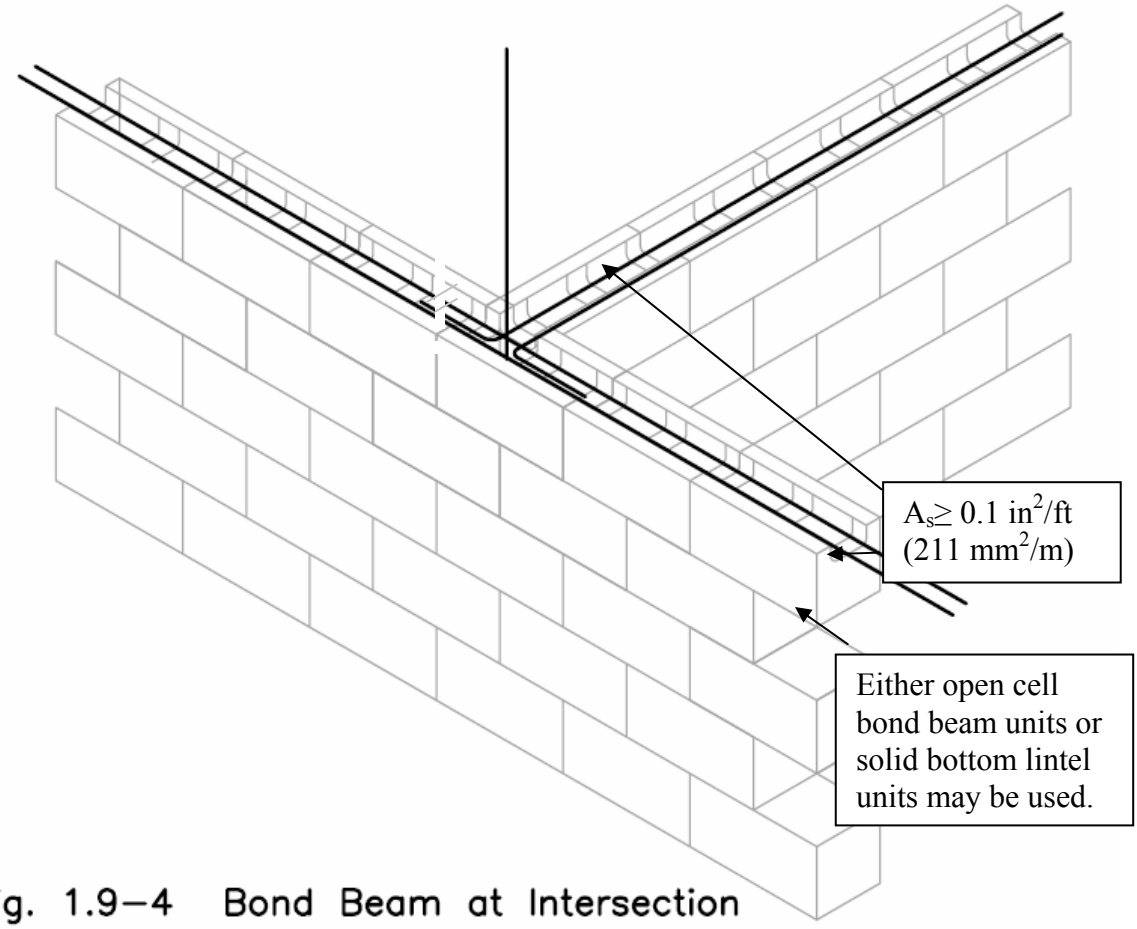
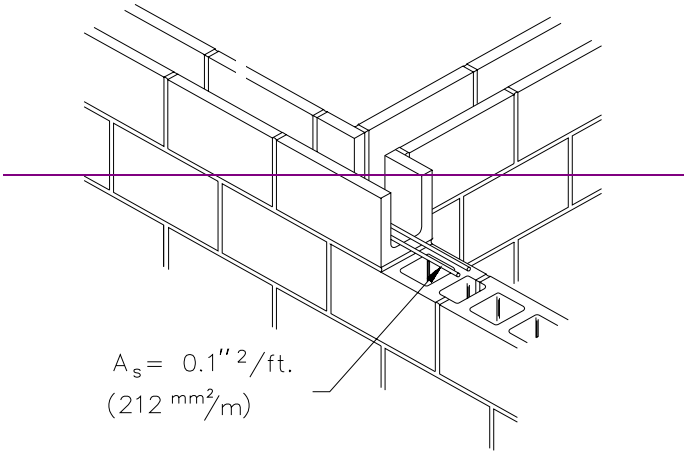


Fig. 1.9–4 Bond Beam at Intersection



~~Fig.~~Figure 1.9-4 — ~~Bonding ties and grouting for flanged shear walls~~Bond beam at wall intersection

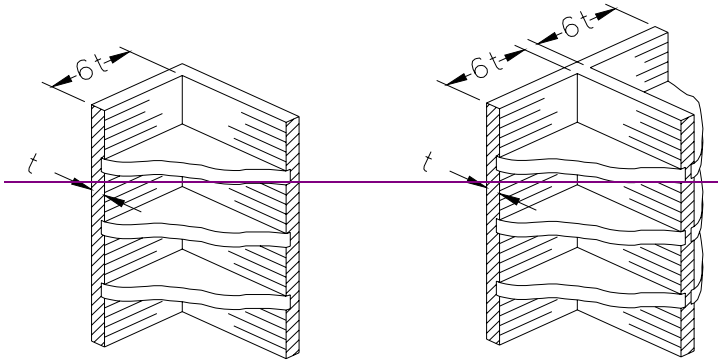


Fig. 1.9-5 — Effective flange width

1.9.5 *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 Fig.Figure 1.9-5. 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.

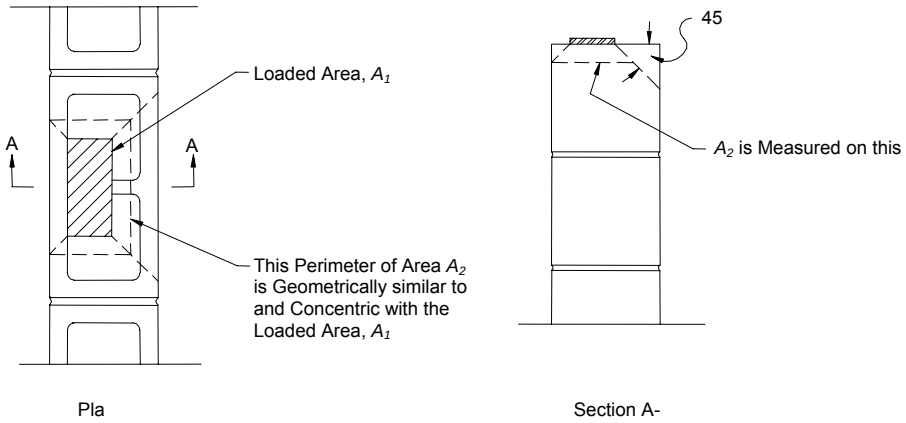
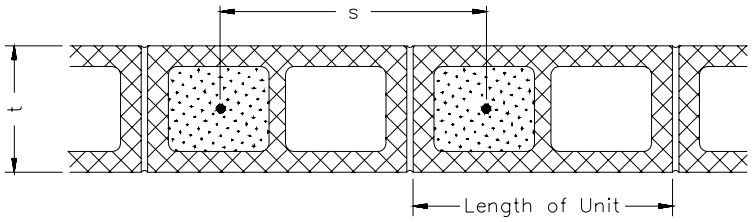


Fig.Figure 1.9-5 — Bearing areas

1.9.6 *Effective compressive width per bar* — The effective width of the compressive area for each reinforcing bar must be established. Fig.Figure 1.9-61.9-6 depicts the limits for the conditions stated. Limited research[1.1415]20 is available on this subject.

The limited ability of head joints to transfer stress when the masonry is laid in stack bond is recognized by the requirements for bond beams. Masonry units with open ends that are solidly grouted are assumed to transfer stress as indicated in Section 2.2.5.2(d), as for running bond.

The center-to-center bar 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.



For masonry in other than running bond with bond beams spaced less than or equal to 48 in. (1219 mm) and running bond masonry, b equals the lesser of:

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

For masonry in other than running bond with bond beams spaced greater than 48 in. (1219 mm), b equals the lesser of:

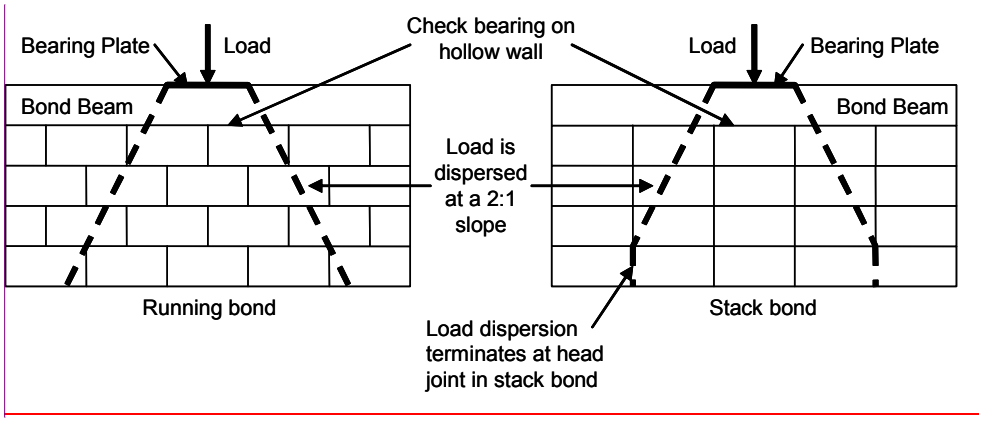
b = s
b = length of unit

~~*Fig.*~~*Figure* ~~*1.9-*~~*1.9-*~~*66*~~ — *Width of compression area*

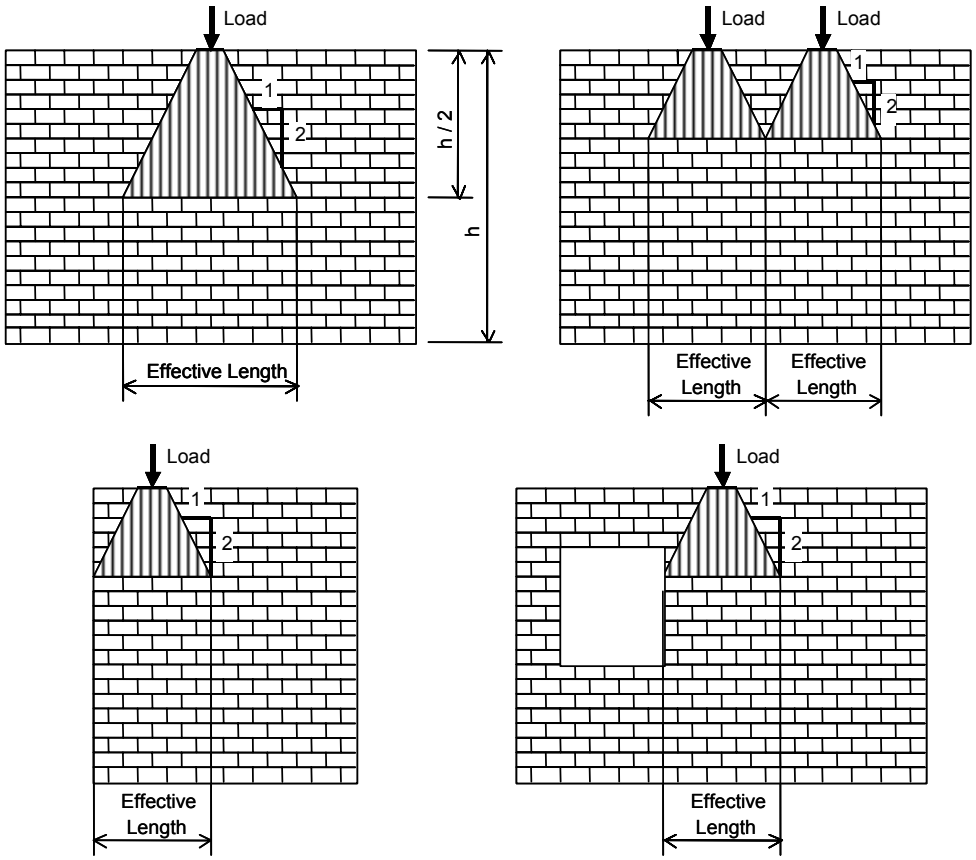
1.9.7 *Concentrated loads* — Reference 1.21 reports the results of tests of a wide variety of specimens under concentrated loads, including AAC masonry, concrete block masonry, and clay brick masonry specimens. Reference 1.21 suggests that a concentrated load can be distributed at a 2:1 slope, terminating at half the wall height, where the wall height is from the point of application of the load to the foundation. Tests on the load dispersion through a bond beam on top of hollow masonry reported in Reference 1.22 resulted in an angle from the horizontal of 59º for a 1-course CMU bond beam, 65º for a 2-course CMU bond beam, and 58º 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.

The illustration of the code provisions is shown in Figure 1.9-7. Figure 1.9-7a illustrates the dispersion of a concentrated load through a bond beam for both running bond and stack bond. A

hollow wall would be checked for bearing under the bond beam using the effective length. Figure 1.9-7b illustrates the dispersion of a concentrated load in the wall. The effective length would be used for checking the wall under the axial force. A wall may have to be checked at several locations, such as under a bond beam and at midheight.



(a) Distribution of concentrated load through bond beam



(b) Distribution of concentrated load in wall

Figure 1.9-7. Distribution of concentrated loads

1.10 — Deflection

1.10.1 *Deflection of beams and lintels*

~~These deflection limits apply to beams of all materials that support unreinforced masonry.~~

~~These empirical requirements limit excessive deflections that may result in damage to the supported masonry. Where supported masonry is designed in accordance with Section 2.3, it is assumed that crack width in masonry will be controlled by the reinforcement so the deflection requirements are waived.~~

The provisions of Section 1.10.1 address deflections that may occur at service load levels.

1.10.1.1 The deflection limits apply to beams and lintels of all materials that support unreinforced masonry. The deflection requirements may also be applicable to supported reinforced masonry that has vertical reinforcement only.

Traditional deflection limits for masonry are $l/600$ or 0.3 in. (7.6 mm). In most cases, deflections of approximately twice this amount, or $l/300$, are required before the deflection becomes visible[1.21]1.23. The traditional limit of $l/600$ should prevent long-term visible deflections and serviceability problems. This deflection limit is for immediate deflections. Creep will cause additional long term deflections. A larger deflection limit of $l/480$ has been used when considering long term deflections[1.22]1.24.

1.10.1.2 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[1.23]1.25. This procedure was selected as being sufficiently accurate for use to control deflections[1.24]1.26. 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 ~~anywhere~~ from 20 to 45% of those of the simply supported specimens[1.25]1.27.

1.10.1.3 Reinforced masonry beams and lintels with span lengths of 8 times d have immediate deflections of approximately 1/600 of the span length. Masonry beams and lintels with shorter spans will have sufficient stiffness to prevent serviceability problems and therefore deflections do not need to be checked.

1.10.2 *Connection to structural frames*

Exterior masonry walls connected to structural frames are used primarily as non-bearing curtain walls. Regardless of the structural system used for support, there are differential movements between the structure and the wall. These differential movements may occur separately or in combination and may be 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.

Since the tensile strength of masonry is low, these differential movements must be accommodated by sufficient clearance between the frame and masonry and flexible or slip-type connections.

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.

Wood, steel, or concrete columns may be surrounded by masonry serving as a decorative element. Masonry walls may be subject to forces as a result of their interaction with other structural components. Since the masonry element is often much stiffer, the load will be carried primarily by the masonry. These forces, if transmitted to the surrounding masonry, should not exceed the allowable stresses of the masonry. Alternately, there should be sufficient clearance between the frame and masonry. Flexible ties should be used to allow for the deformations.

Beams or trusses supporting masonry walls are essentially embedded, and their deflections should be limited to the allowable deflections for the masonry being supported. See Section 1.10.1 for requirements.

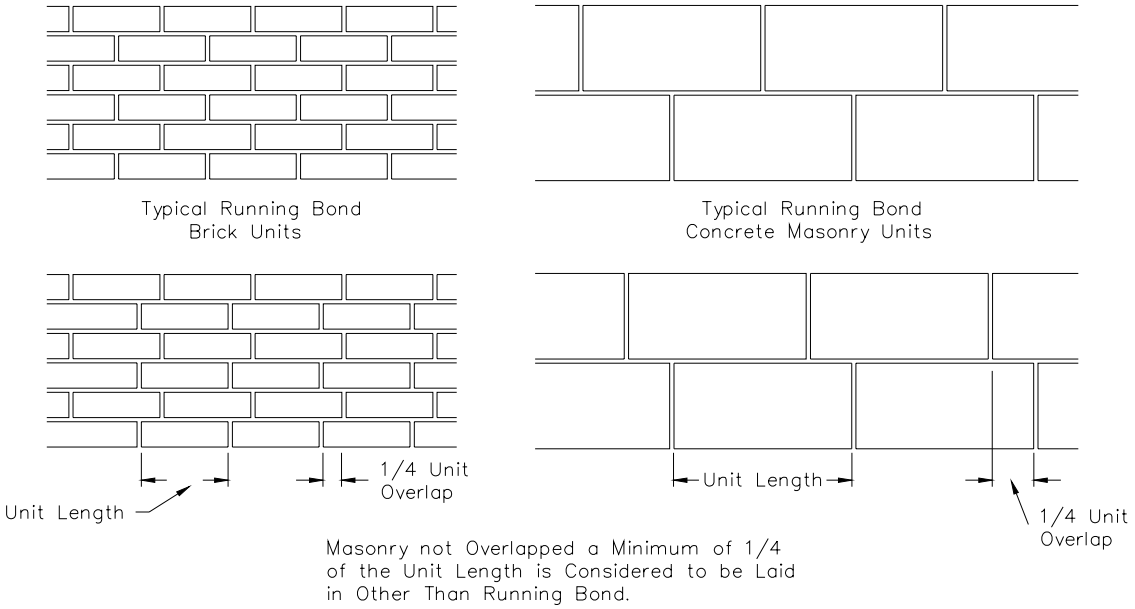
1.11 — Stack bond masonry

The requirements separating running bond from stack bond are shown in ~~Fig.~~Figure 1.11-1. The amount of steel required in this section is ~~an arbitrary~~a prescriptive amount to provide continuity across the head joints. This reinforcement can be also used to resist load.

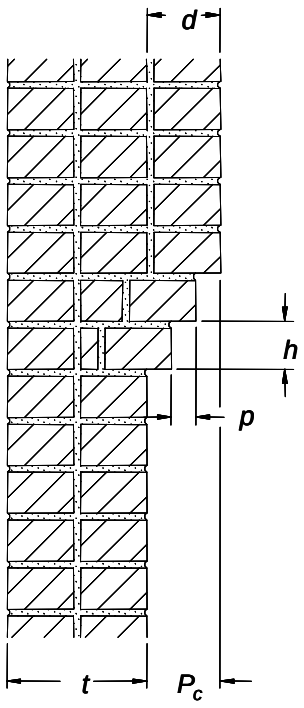
Although continuity across head joints in masonry laid in other than running bond is a concern for AAC masonry as well as masonry of clay or concrete, the use of horizontal reinforcement to enhance continuity is generally practical only by the use of bond beams.

1.12 — Corbels

The provision for corbelling up to one-half of the 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-mm) intrusion into the plane recognizes the impracticality of keeping the back surface plane. See ~~Fig.~~Figure 1.12-1 and 1.12-2 for maximum permissible unit projection.



~~Fig.~~ *Figure 1.11-1 — Running bond masonry*



Limitations on Corbelling:

$P_c \leq t/2$

$p \leq h/2$

$p \leq d/3$

Where:

P_c = Allowable total horizontal projection of corbelling

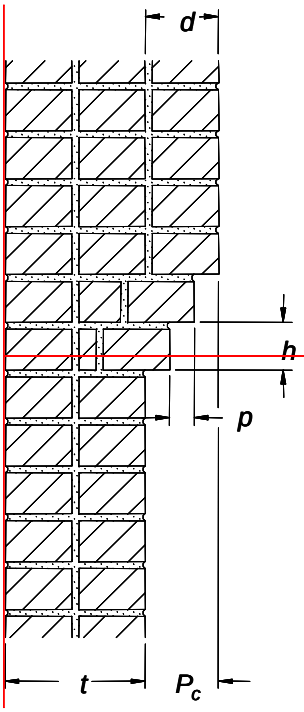
p = Allowable projection of one unit

t = nominal wall thickness

d = nominal unit thickness (specified thickness plus the thickness of one mortar joint)

h = nominal unit height (specified height plus the thickness of one mortar joint

Note: Neither ties nor headers shown.



Limitations on Corbelling:

$P_C = t / 2$

$p = h / 2$

$p = d / 3$

Where:

P_C = Allowable total horizontal projection of corbelling

p = Allowable projection of one unit

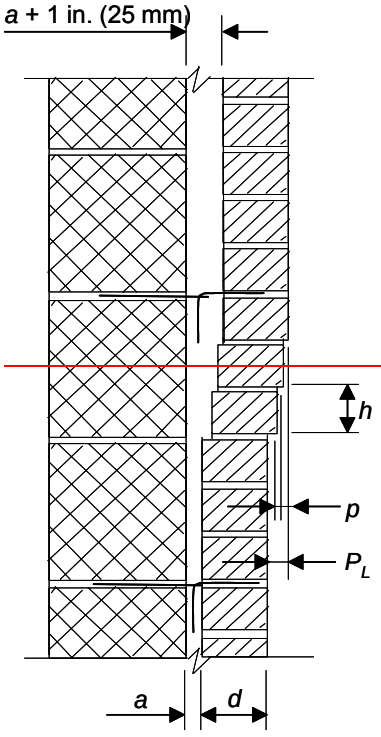
t = Nominal wall thickness

d = Nominal unit thickness (specified thickness plus the thickness of one mortar joint)

h = Nominal unit height (specified height plus the thickness of one mortar joint)

a = Air space thickness

~~Fig.~~ Figure 1.12-1 — Limits on corbelling in solid walls



Limitations on Corbelling:

$P_C = d / 2$

$p = h / 2$

$p = d / 3$

Where:

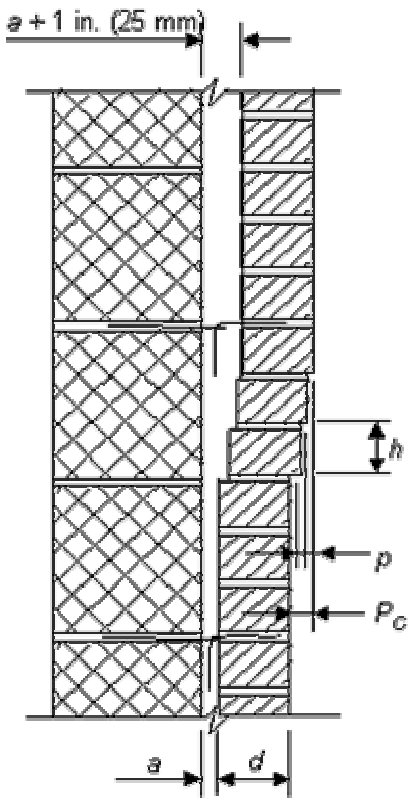
P_C = Allowable total horizontal projection of corbelling

p = Allowable projection of one unit

d = Nominal unit thickness (specified thickness plus the thickness of one mortar joint)

h = Nominal unit height (specified height plus the thickness of one mortar joint)

a = Air space thickness



Limitations on Corbelling:

$P_C \leq d/2$

$p \leq h/2$

$p \leq d/3$

Where:

P_C = Allowable total horizontal projection of corbelling

p = Allowable projection of one unit

d = Nominal unit thickness (specified thickness plus the thickness of one mortar joint)

h = Nominal unit height (specified height plus the thickness of one mortar joint)

a = Air space thickness

Ties shown for illustration only

~~Fig.~~ Figure 1.12-2 – Limits on corbelling in walls with air space

1.13 — Beams

1.13.2 *Lateral Support*

To minimize lateral torsional buckling, the Code requires lateral bracing of the compression face. The requirement applies to simply supported beams as written. With continuous or fixed beams, the spacing may be increased.

1.14 — Columns

Columns are defined in Section 1.6. 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 elements warrant the special requirements of this section ~~that were selected after extensive committee consideration~~.

1.14.1 *General column design*

1.14.1.1 *Dimensional limits* — The minimum nominal side dimension of 8 in. (203 mm) results from practical considerations.

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

1.14.1.3 *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 shear1.~~2~~2682. Ties may be located in the mortar joint, when the tie diameter does not exceed ½ the specified mortar joint thickness. For example, ¼ in. (6.4 mm) diameter ties may be placed in ½ in. (12.7 mm) thick mortar joints.

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

1.14.2 *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 1.14.1. The axial load limit of 2000 pounds (8900 N) was developed based on the flexure capacity of a nominal 8 in. (203 mm) by 8 in. (203 mm) by 12 ft high (3.66 m) column with one No. 4 reinforcing bar in the center and f'_m of 1350 psi (9.31 MPa). An axial load of 2000 pounds (8900 N) at the edge of the member will result in a moment that is approximately equal to the moment capacity of this member.

1.131.15 — Details of reinforcement and metal accessories

When the provisions of this section were originally developed in the late 1980's, the Committee used the then current ACI 318 Code[1.14-15162997] as a guide. Some of the requirements were simplified and others dropped, depending on their suitability for application to masonry.

1.131.15.2 *Size of reinforcement*

1.131.15.2.1 Limits on size of reinforcement are based on accepted practice and successful performance in construction. The No. 11 (M#36) limit is arbitrary, but Reference 2.502.46 shows that distributed small bars provide better performance than fewer large bars. Properties of reinforcement are given in Table 1.131.15.2.

1.131.15.2.2 Adequate flow of grout necessary for good bond is achieved with this limitation. It also limits the size of reinforcement when combined with Section 1.16.11.181.19.1.

1.131.15.2.3 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.[1.2830156 17] Joint reinforcement is commonly used in concrete masonry to minimize shrinkage cracking. 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.

1.131.15.3 *Placement of 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. A minimum spacing between bars in a layer prevents longitudinal splitting of the masonry in the plane of the bars. Use of bundled bars in masonry construction is rarely required. Two bars per bundle is considered a practical maximum. It is important that bars be placed accurately. Reinforcing bar positioners are available to control bar position.

~~1.13~~1.15.4 *Protection of reinforcement*

~~1.13~~1.15.4.1 Reinforcing bars are traditionally not galvanized. The masonry cover retards corrosion of the steel. Cover is measured from the exterior masonry surface to the outer-most surface of the steel to which the cover requirement applies. It is measured to the outer edge of stirrups or ties, if transverse reinforcement encloses main bars. 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 outer-most surface of the steel.

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.

~~1.13~~1.15.4.2 Since masonry cover protection for joint reinforcement is minimal, the protection of joint reinforcement in masonry is required in accordance with the Specification. Examples of interior walls exposed to a mean relative humidity exceeding 75 percent are natatoria and food processing plants.

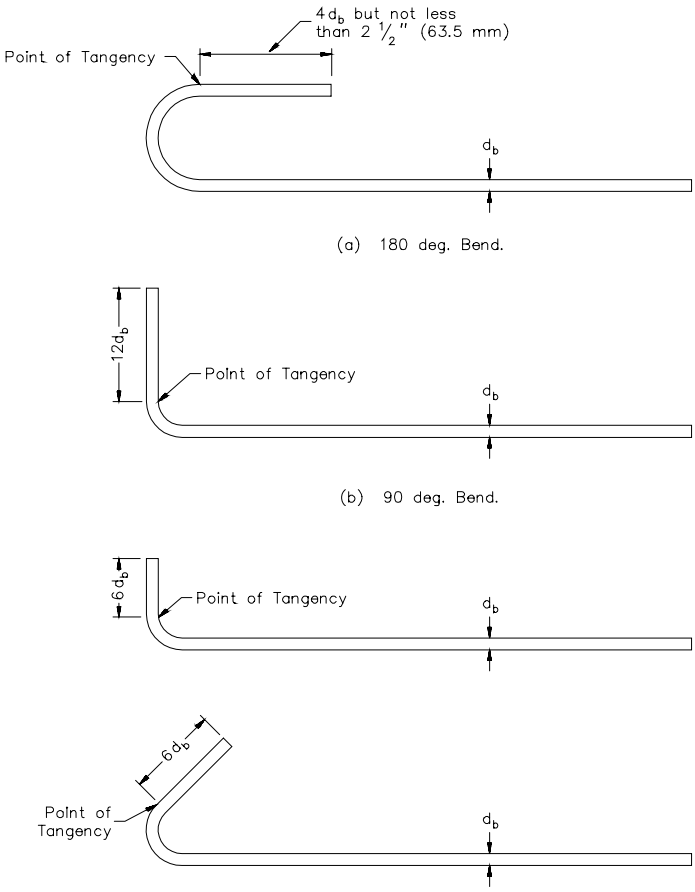
~~1.13~~1.15.4.3 Corrosion resistance requirements are included since masonry cover varies considerably for these items. The exception for anchor bolts is based on current industry practice.

Table ~~1.13~~1.15.2 — Physical properties of steel reinforcing wire and bars

Designation	Diameter, in. (mm)	Area, in.^{2} (mm^2)	Perimeter, in. (mm)
Wire			
W1.1 (11 gage) (MW7)	0.121 (3.1)	0.011 (7.1)	0.380 (9.7)
W1.7 (9 gage) (MW11)	0.148 (3.8)	0.017 (11.0)	0.465 (11.8)
W2.1 (8 gage) (MW13)	0.162 (4.1)	0.020 (12.9)	0.509 (12.9)
W2.8 (3/16 wire)	0.187 (4.8)	0.027 (17.4)	0.587 (14.9)
~~(MW17)~~(MW18)	0.250 (6.4)	0.049 (31.6)	0.785 (19.9)
W4.9 ($^1/_4$ wire) (MW32)			
Bars			
No. 3 (M#10)	0.375 (9.5)	0.11 (71.0)	1.178 (29.9)
No. 4 (M#13)	0.500 (12.7)	0.20 (129)	1.571 (39.9)
No. 5 (M#16)	0.625 (15.9)	0.31 (200)	1.963 (49.9)
No. 6 (M#19)	0.750 (19.1)	0.44 (284)	~~2.456~~
No. 7 (M#22)	0.875 (22.2)	0.60 (387)	~~(62.4)~~2.356
No. 8 (M#25)	1.000 (25.4)	0.79 (510)	(59.8)
No. 9 (M#29)	1.128 (28.7)	1.00 (645)	2.749 (69.8)
No. 10 (M#32)	1.270 (32.3)	1.27 (819)	3.142 (79.8)
No. 11 (M#36)	1.410 (35.8)	1.56 (1006)	3.544 (90.0)
			3.990 (101)
			4.430 (113)

~~1.13~~1.15.5 *Standard hooks*

Standard hooks are shown in ~~Fig.~~Figure ~~1.13~~1.15-1.



~~Fig.~~Figure ~~1.13~~1.15-1 — Standard hooks

~~1.13~~1.15.6 *Minimum bend diameter for reinforcing bars*

Standard bends in reinforcing bars are described in terms of the inside diameter of bend since 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 (413.7 MPa) 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.

1.16 *Anchor bolts* - These design values apply only to the specific types of bolts mentioned. These bolts are readily available and are depicted in Figure 1.16-1.

1.16.1 *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.

1.16.2 *Projected area for axial tension* – Results of tests[1.31,1.32] on headed anchor bolts in tension showed that anchors 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 45º in the direction of the pull (See Figure 1.16-2). 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 1.16-3. 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}.

1.16.3 *Projected area for shear* – Results of tests[1.31,1.32] on anchor bolts in shear showed that anchors 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 45º in the direction of the pull towards the free edge of the masonry (See Figure 1.16-4). Pryout (See Figure 1.16-5), 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 1.16.2.

1.16.5 *Effective embedment length for bent-bar anchor bolts* – Tests[1.31] have shown that the pullout strength of bent-bar anchors correlated best with a reduced embedment length. This may be explained with reference to Figure 1.16-6. Due to the radius of the bend, stresses are concentrated at a point less than the full embedment length.

1.16.6 *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.

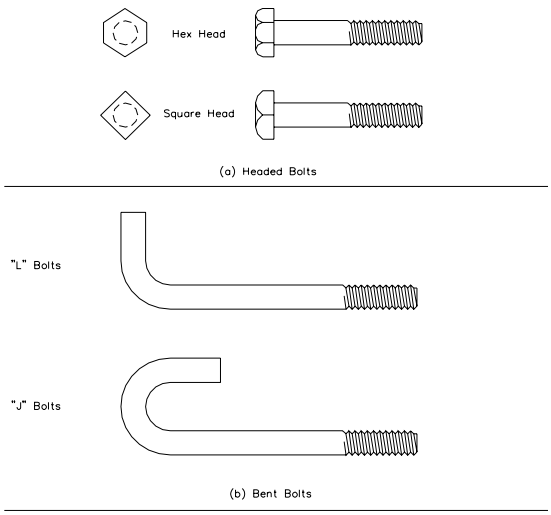


Figure 1.16-1 — Anchor bolts

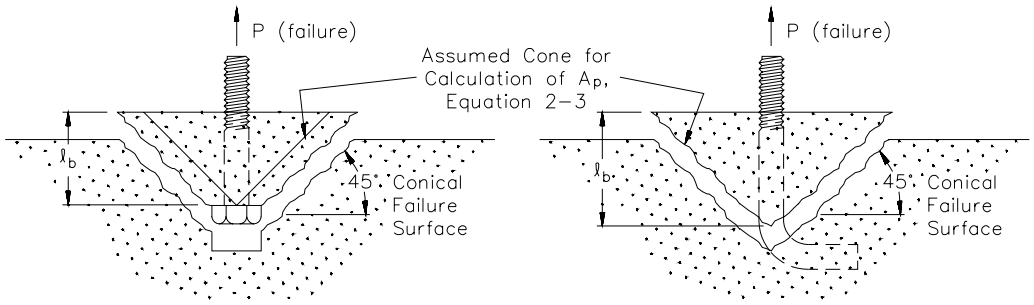


Figure 1.16-2 — Anchor bolt tensile breakout cone (Staff to change Ap to Apt)

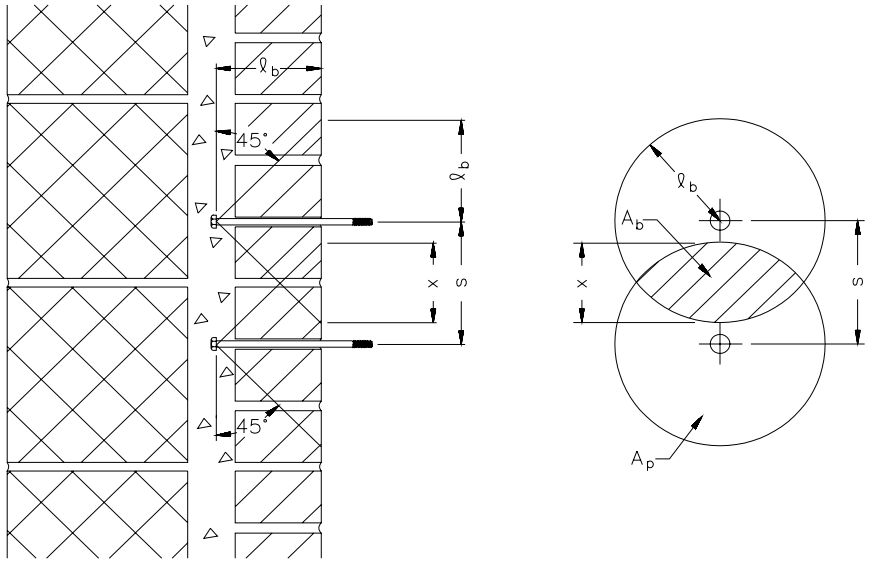


Figure 1.16-3 — Overlapping anchor bolt breakout cones Staff to change Ap to Apt

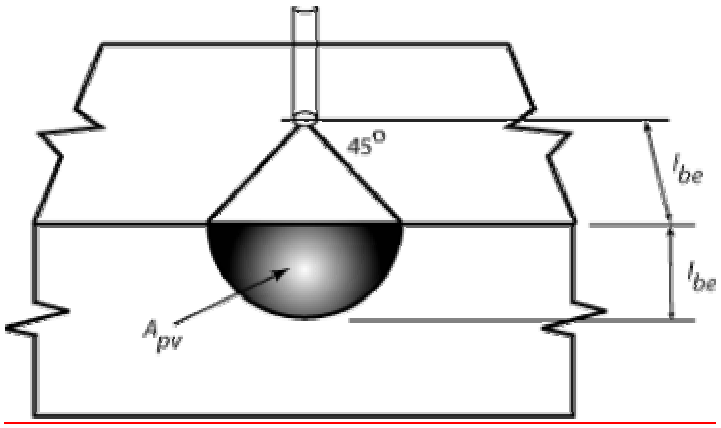


Figure 1.16-4- Anchor bolt shear breakout (Change back solid line to broken line) *Staff to recreate figure*

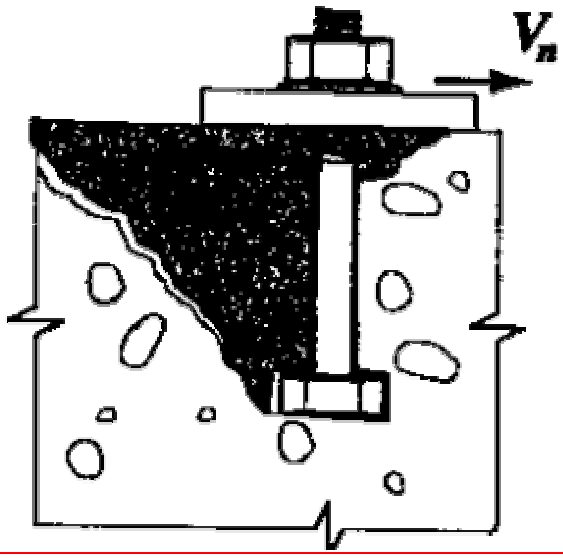


Figure 1.16-5 - Anchor bolt shear pryout Staff to recreate figure

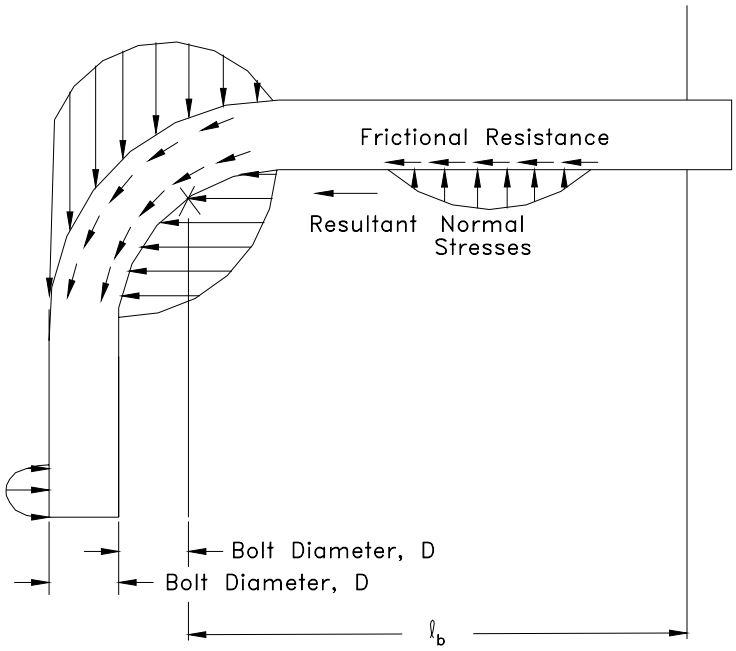


Figure 1.16-6 — Stress distribution on bent anchor bars Change D to db

1.141.161.17 — Seismic design requirements

1.141.161.17.1 *Scope*

The requirements in this section have been devised to improve performance of masonry construction when subjected to earthquake loads. ASCE 7-02 has been cited here as the appropriate reference for the distribution of seismic forces in order to avoid confusion in the event that the legally adopted building code has no provisions or is inconsistent with the type of distribution upon which these provisions are based.

The special provisions 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.

Seismic requirements for masonry veneers are given in Chapter 6, Veneers.

1.141.161.17.2 *General*

By reference to Section 1.1.3, the designer is permitted to use allowable stress design methods for reinforced masonry, allowable stress design for unreinforced masonry, allowable stress design for prestressed masonry with noted modifications, or empirical design. The alternate method in Section 2.1.3.4 permits a strength design methodology in which allowable stress values are modified to approximate strength value levels. The designer should note that the limitations of the Seismic Design Categories may further limit the available design options. For instance, empirical design procedures are not permitted to be used for structures in Seismic Design Categories D, E, and F. Chapter 5, Empirical Design of Masonry, does not permit empirical design for the lateral force-resisting system in Seismic Design Categories B and C.

If the legally adopted building code has adopted the seismic load provisions of ASCE 7-02, the "strength" design procedures of Section 2.1.3 should be used. If the legally adopted building code has seismic load provisions specifically intended for working stress design, the allowable stress design procedures of Chapter 2 should be used. The Architect/Engineer should be aware that the use of "strength" level loads should not be used in conjunction with allowable stress design procedures as overly conservative design can result. Similarly, the use of "allowable stress" loads in conjunction with strength design procedures could result in unconservative designs.

1.141.161.17.2.2 *Lateral force-resisting system* — A lateral force-resisting system must be defined for all buildings. Most masonry buildings use masonry shear walls to serve as the lateral 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 Chapter 2, 3, or 4, unless the structure is assigned to Seismic Design Category A, in which case empirical provisions of Chapter 5 may be used.

Five shear wall types are defined, each intended to have a different capacity for inelastic response and energy dissipation in the event of a seismic event. These five shear wall types are assigned different system design parameters such as response modification ~~factors~~coefficients, *R*, based on their expected performance and ductility. Certain shear wall types are permitted in each seismic design category, and unreinforced shear wall types are not permitted in regions of intermediate and high seismic risk. Table ~~1.14~~1.16~~1.16~~1.17.2 summarizes the requirements of each of the five types of masonry shear walls:

~~1.14~~1.16~~1.16~~1.17.2.2.1 *Ordinary plain (unreinforced) 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 ~~1.14~~1.16~~1.16~~1.17.2 Requirements for Masonry Shear Walls based on Shear Wall Designation

Shear wall Designation	Design Methods	Reinforcement Requirements	May be Used In
Empirical Shear Wall	Section 5.3	None	SDC A
Ordinary Plain (Unreinforced) Masonry Shear Wall	Section 2.2, Section 3.2, or Chapter 4	None	SDC A & B
Detailed Plain (Unreinforced) Masonry Shear Walls	Section 2.2 or Section 3.2	Section ~~1.14~~1.16~~1.16~~1.17.2.2.2.1 and ~~1.14~~1.16~~1.16~~1.17.2.2.2.2	SDC A & B
Ordinary Reinforced Masonry Shear Walls	Section 2.3 or Section 3.3	Section ~~1.14~~1.16~~1.16~~1.17.2.2.2.1 and ~~1.14~~1.16~~1.16~~1.17.2.2.2.2	SDC A, B & C
Intermediate Reinforced Masonry Shear Walls	Section 2.3 or Section 3.3	Section ~~1.14~~1.16~~1.16~~1.17.2.2.4	SDC A, B & C
Special Reinforced Masonry Shear Walls	Section 2.3 or Section 3.3	Section ~~1.14~~1.16~~1.16~~1.17.2.2.5	SDC A, B, C, D, E & F

~~1.14~~1.16~~1.16~~1.17.2.2.2 *Detailed plain (unreinforced) masonry shear walls* — These shear walls are designed as plain (unreinforced)) masonry per the sections noted, but contain minimum

reinforcement in the horizontal and vertical directions. Because of this reinforcement, these walls have more favorable seismic design parameters, including higher response modification ~~factors~~coefficients, *R*, than ordinary plain (unreinforced) masonry shear walls.

1.14~~1.16~~1.17.2.2.2.1 *Minimum reinforcement requirements* — The provisions of this section require a judgment-based minimum amount of reinforcement to be included in masonry wall construction. Tests reported in Reference 1.~~1~~6~~7~~18~~29~~33 have confirmed that masonry construction, reinforced as indicated, performs adequately at this seismic load level. This minimum required reinforcement may also be used to resist design loads.

1.14~~1.16~~1.17.2.2.2.2 *Connections* — 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. 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.

1.14~~1.16~~1.17.2.2.3 *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 (unreinforced) 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~~coefficients, *R*, than plain (unreinforced) masonry shear walls. When assigned to moderate seismic risk areas (Seismic Design Category C), however, minimum reinforcement is required as noted in Section ~~1.14~~1.~~16~~1.17.2.2.2.1.

1.14~~1.16~~1.17.2.2.4 *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 (unreinforced) 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~~coefficients, *R*, than plain (unreinforced) masonry shear walls and ordinary reinforced masonry shear walls.

~~1.14~~1.~~16~~1.17.2.2.5 *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 in all seismic risk areas. Additionally, these walls have the most favorable seismic design parameters, including the highest response modification ~~factor~~coefficient, *R*, of any of the masonry shear wall types. The intent of Sections ~~1.14~~1.~~16~~1.17.2.2.5(a) through ~~1.14~~1.~~16~~1.17.2.2.5(c) is to provide a minimum level of in-plane shear reinforcement to improve ductility.

~~1.14~~1.~~16~~1.17.2.2.7 *Detailed plain (unreinforced) 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 (References A~~8~~.3 and A~~8~~.1). The maximum distance from the edge of an opening or end of a wall to the vertical reinforcement is set at 24 inches (610 mm)since the typical length of an AAC unit is 24 inches (610 mm).

~~1.14~~1.~~16~~1.17.3 *Seismic Design Category A*

The general requirements of this Code provide for adequate performance of masonry construction in areas of low seismic risk.

~~1.14~~1.~~16~~1.17.4 *Seismic Design Category B*

Although masonry may be designed by the provisions of Chapter 2, Allowable Stress Design; Chapter 3, Strength Design of Masonry; Chapter 4, Prestressed Masonry; or Chapter 5, Empirical Design of Masonry, the lateral force-resisting system for structures in Seismic Design Category B must be designed based on a structural analysis in accordance with Chapter 2, 3, or 4. The provisions of Chapter 5 cannot be used to design the lateral force-resisting system of buildings in Seismic Design Category B.

~~1.14~~1.~~16~~1.17.4.2 *Design of elements that are part of the lateral force-resisting system* — As a minimum, shear walls in masonry structures assigned to Seismic Design Category B are required to comply with the requirements of ordinary plain (unreinforced), detailed plain (unreinforced), ordinary reinforced, intermediate reinforced, or special reinforced masonry shear

walls. Masonry shear walls are required to designed by either Chapter 2 or Chapter 3 in Seismic Design Categories B and higher.

1.141.161.17.5 *Seismic Design Category C*

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.

1.141.161.17.5.3.1 *Connections to masonry columns* — 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.

1.141.161.17.5.3.2 *Masonry shear walls* — Masonry shear walls for structures assigned to Seismic Design Category C are required to be reinforced because of the increased risk and expected intensity of seismic activity. Ordinary reinforced masonry shear walls, intermediate reinforced masonry shear walls, or special reinforced masonry shear walls are required to be used

1.141.161.17.5.3.3 *Anchorage of floor and roof diaphragms in AAC masonry structures* — In Seismic Design Categories C and D additional connectors are required, with the intention of ensuring ductile behavior.

1.141.161.17.6 *Seismic Design Category D*

1.141.161.17.6.3 *Minimum reinforcement requirements for masonry walls* — The minimum amount of wall reinforcement 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. Since 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 in both directions.

1.141.161.17.6.4 *Masonry shear walls* — Masonry shear walls for structures assigned to Seismic Design Category D are required to meet the requirements of special reinforced masonry shear walls because of the increased risk and expected intensity of seismic activity.

1.141.161.17.6.5 *Minimum reinforcement for masonry columns* — Adequate lateral restraint is important for column reinforcement subjected to overturning forces due to earthquakes.

Many column failures during earthquakes have been attributed to inadequate lateral tying. For this reason, closer spacing of ties than might otherwise be required is prudent. An arbitrary minimum spacing has been established through experience. Columns not involved in the lateral force-resisting system should also be more heavily tied at the tops and bottoms for more ductile performance and better resistance to shear.

~~1.14~~1.~~16~~1.17.7 *Seismic Design Categories E and F*

See Commentary Sections ~~1.14~~1.~~16~~1.17.2.2.2.1 and ~~1.14~~1.~~16~~1.17.6.3. The ratio of minimum horizontal reinforcement is increased to reflect the possibility of higher seismic loads. Where solidly 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.

~~1.15~~1.~~17~~1.18 — Quality assurance program

The allowable values for masonry design permitted by this Code are valid when the quality of masonry construction meets or exceeds that described in the Specification. 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.

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

Section ~~1.15~~1.~~17~~1.18 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 an essential facility or not, as defined by ASCE 7~~-02~~ or the legally adopted building code. Because of their importance, essential facilities are subjected to more extensive~~greater~~ quality assurance measures.

The level of required quality assurance depends on whether the masonry was designed in accordance with Chapters 2, 3, or 4 or Appendix A (engineered) or in accordance with Chapters 5, 6, or 7 (empirical or prescriptive).

1.151.171.18.4 Premixed mortars and grouts are delivered to the project as "trowel ready" or "pourable" materials, respectively. Preblended mortars and grouts are dry combined materials that are mixed with water at the 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.

1.151.171.18.5 In addition to specifying testing and 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 the design/construct team, all 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 that all each party is ies are informed and that records for documenting construction occurrences are available and correct after construction has been completed.

~~1.15~~1.~~17~~1.18.6 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.

Laboratories that comply with the requirements of ASTM C 1093[1.~~21~~~~22~~~~23~~~~40~~] are more likely to be familiar with masonry materials and testing. Specifying that the testing agencies comply with the requirements of ASTM C 1093 should improve the quality of the resulting masonry.

~~1.15.7~~1.~~17~~1.18.7 *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 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 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.

~~1.16~~1.~~18~~1.19 — Construction

The ACI 530.1/ASCE 6/TMS 602 Specification covers material and construction requirements. It is an integral part of the Code in terms of minimum requirements relative to the composition, quality, storage, handling, and placement of materials for masonry structures. The Specification also includes provisions requiring verification that construction achieves the quality specified. The construction must conform to these requirements in order for the Code provisions to be valid.

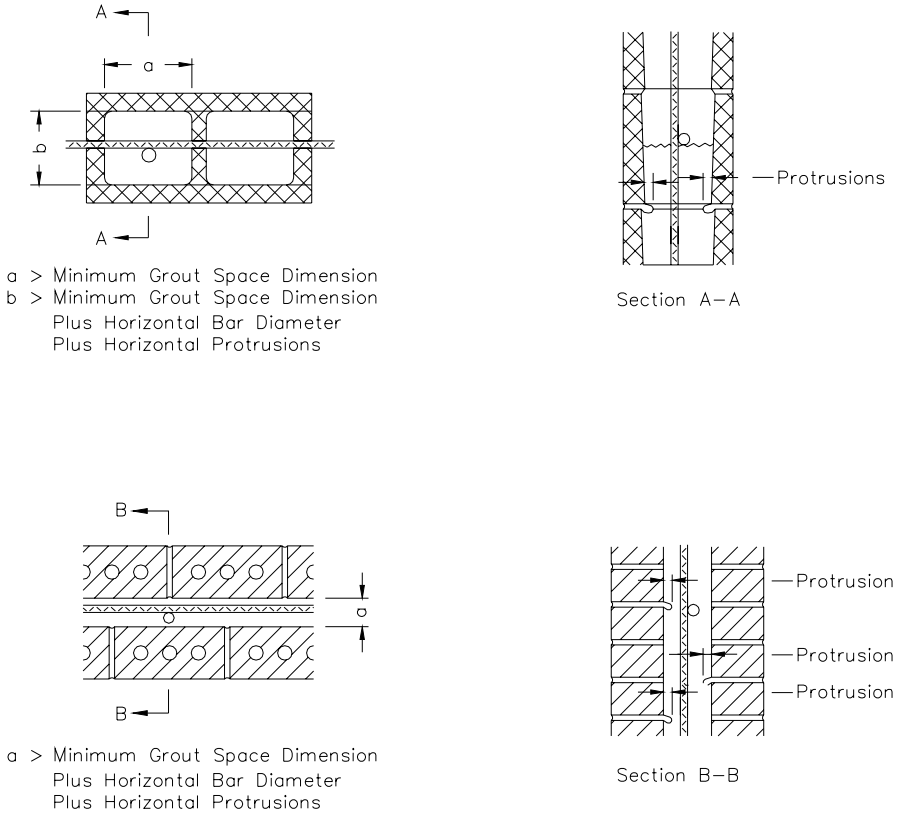
~~1.16.1~~1.~~18~~1.19.1 *Grouting, minimum spaces*

Code Table ~~1.16.1~~1.~~18~~1.19.1 contains the least clear dimension for grouting between wythes and the minimum cell dimensions when grouting hollow units. Selection of units and bonding pattern should be coordinated to achieve these requirements. Vertical alignment of cells must also be considered. ~~All p~~ Projections or obstructions into the grout space and the diameter of horizontal

reinforcement must be considered when calculating the minimum dimensions. See Fig. Figure 1.16 1.18 1.19-1.

Coarse grout and fine grout are differentiated by aggregate size in ASTM C 476.

The grout space requirements of Code Table 1.16.1 1.18 1.19.1 are based on usual grout aggregate size and cleaning practice to permit the complete filling of grout spaces and adequate consolidation using typical methods of construction. Grout spaces smaller than specified in Table 1.16.1 1.18 1.19.1 have been used successfully in some areas. When the Architect/Engineer is requested to accept a grouting procedure that exceeds the limits in Table 1.16.1 1.18 1.19.1, 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 elements 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.



~~Fig.~~Figure ~~1.16~~1.18~~1~~1.19-1 — Grout space requirements

~~1.16.2~~1.18~~1~~1.19.2 *Embedded conduits, pipes, and sleeves*

~~1.16.2~~1.18~~1~~1.19.2.1 Conduits, pipes, and sleeves not harmful to mortar and grout can be embedded within the masonry, but the capacity of the wall should not be less than that required by design. The effects of a reduction in section properties in the areas of pipe embedment should be considered. Horizontal pipes located in the planes of walls may affect the wall's load capacity.

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.

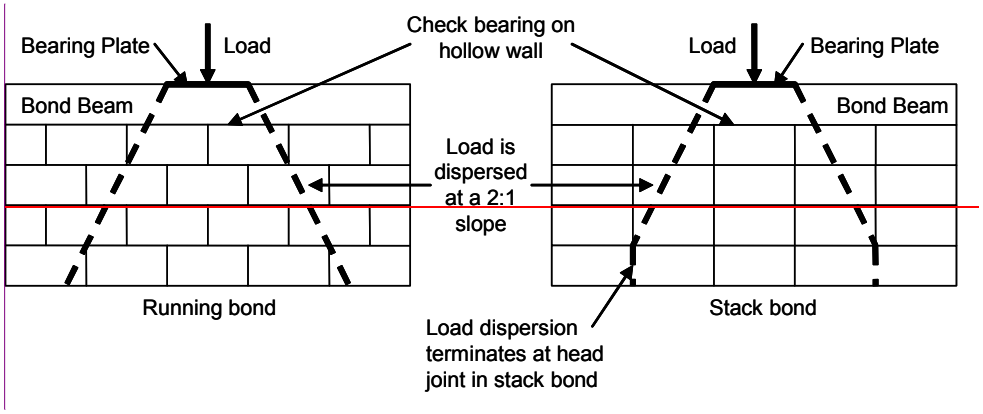
Aluminum should not be used in masonry unless it is effectively coated or covered. Aluminum reacts with ions, and may also react electrolytically with steel, causing cracking and/or spalling of the masonry. Aluminum electrical conduits present a special problem since stray electric current accelerates the adverse reaction.

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

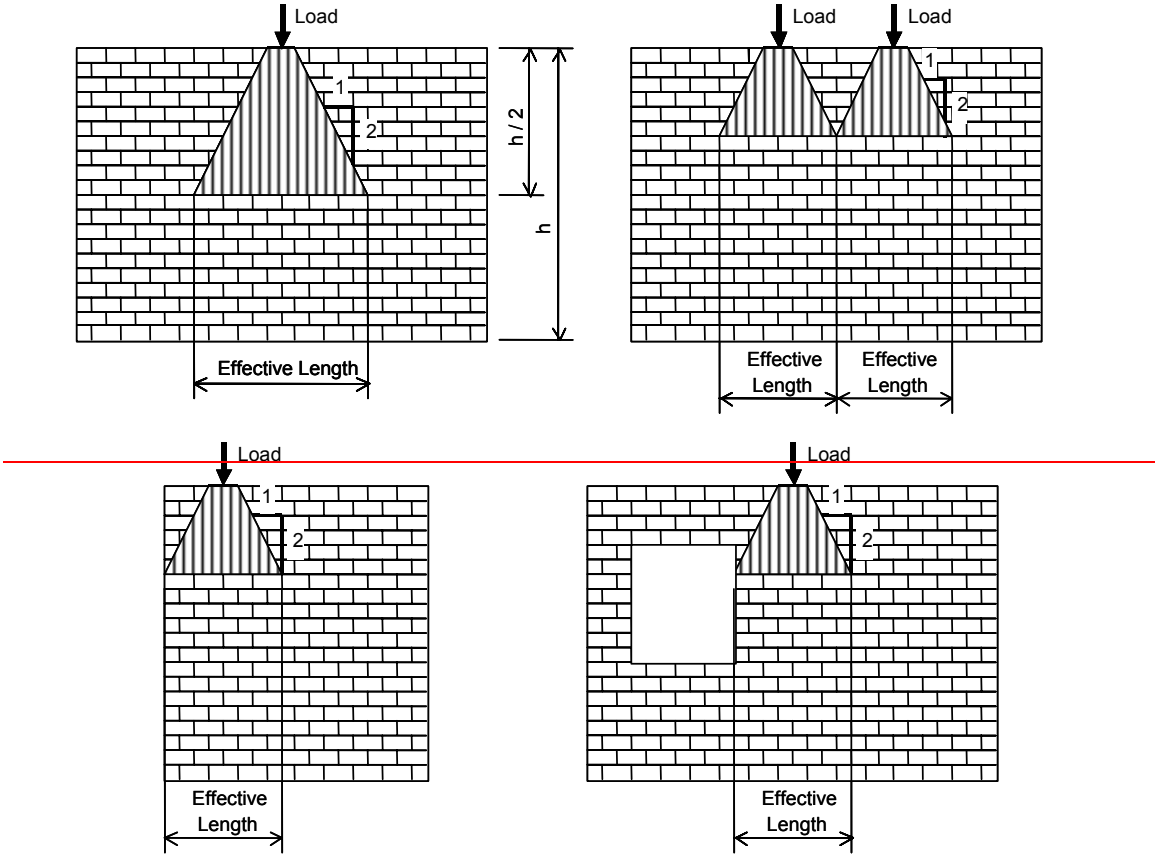
1.19 — *Concentrated loads*

Reference 1.31 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. Reference 1.31 suggests that a concentrated load can be distributed at a 2:1 slope, terminating at half the wall height, where the wall height is from the point of application of the load to the foundation. Tests on the load dispersion through a bond beam on top of hollow masonry reported in Reference 1.32 resulted in an angle from the horizontal of 59º for a 1-course CMU bond beam, 65º for a 2-course CMU bond beam, and 58º 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.

The illustration of the code provisions is shown in Figure 1.19-1. Figure 1.19-1a illustrates the dispersion of a concentrated load through a bond beam for both running bond and stack bond. A hollow wall would be checked for bearing under the bond beam using the effective length. Figure 1.19-1b illustrates the dispersion of a concentrated load in the wall. The effective length would be used for checking the wall under the axial force. A wall may have to be checked at several locations, such as under a bond beam and at midheight.



(a) Distribution of concentrated load through bond beam



(b) Distribution of concentrated load in wall

Fig.Figure 1.19-1. Distribution of concentrated loads

References

1.1. "Glossary of Terms Relating to Brick Masonry," Technical Notes on Brick Construction, No. 2 (Revised), Brick Institute of America, Reston, VA, 1988, 4 pp.

1.2. "Glossary of Concrete Masonry Terms," NCMA TEK Bulletin No. 145, National Concrete Masonry Association, Herndon, VA, 1985, 4 pp.

1.3. "The Masonry Glossary," International Masonry Institute, Washington, DC, 1981, 144 pp.

1.4. *Structural Design of Tall Concrete and Masonry Buildings*, Monograph on Planning and Design of Tall Buildings, V. CB, Council on Tall Buildings and Urban Habitat/American Society of Civil Engineers, New York, NY, 1978, 960 pp.

1.5. Wolde-Tinsae, A.M., Atkinson, R.H. and Hamid, A.A., "State-of-the-Art: Modulus of Elasticity," 6th North American Masonry Conference. Philadelphia, PA, June 1993, pp. 1209-1220, The Masonry Society, Boulder, CO.

1.6. Colville, J., Miltenberger, M.A., and Wolde-Tinsae (Amde), A.M.. "Hollow Concrete Masonry Modulus of Elasticity," 6th North American Masonry Conference, Philadelphia, PA, June 1993, pp. 1195-1208, The Masonry Society, Boulder, CO.

1.7. Copeland, R.E., "Shrinkage and Temperature Stresses in Masonry," ACI JOURNAL, *Proceedings* V. 53, No. 8, American Concrete Institute, Detroit MI, Feb. 1957, pp. 769-780.

1.8. Plummer, H.C., *Brick and Tile Engineering*, Brick Institute of America, Reston, VA, 1962, 736 pp.

1.9. Grimm, C.T., "Probabilistic Design of Expansion Joints in Brick Cladding," *Proceedings*, V.1, 4th Canadian Masonry Symposium, University of Fredericton, 1986, pp. 553-568.

1.10. Kalouseb, L., "Relation of Shrinkage to Moisture Content in Concrete Masonry Units," *Paper* No. 25, Housing and Home Finance Agency, Washington, DC, 1954.

1.11. *"Autoclaved Aerated Concrete Properties, Testing and Design,"* RILEM Recommended Practice, RILEM Technical Committees 78-MCA and 51-ALC. Edited by: S. Aroni, G.J. de Grood, M.F. Robinson, G. Svanholm and F.H. Wittman, E & FN SPON, London, 1993.

1.~~1~~12. Smith, R. G., "Moisture Expansion of Structural Ceramics – Long Term Unrestrained Expansion of Test Bricks," Journal of the British Ceramic Society, Stoke-on-Trent, England, Jan. 1973, pp. 1-5.

1.~~12~~13. "Crack Control in Concrete Masonry Walls," NCMA TEK 10-1A, National Concrete Masonry Association, Herndon, VA, 2001, 4 pp.

1.~~19~~~~20~~~~21~~14. "Control Joints for Concrete Masonry Walls," NCMA TEK 10-2A, National Concrete Masonry Association, Herndon, VA, 1998, 6 pp.

1.15. "All Weather Concrete Masonry Construction," NCMA TEK 3-1C, National Concrete Masonry Association, Herndon, VA, 2002, 4 pp.

1.16 Lenczner, D., and Salahuddin, J., "Creep and Moisture Movements in Masonry Piers and Walls," *Proceedings*, 1st Canadian Masonry Symposium, University of Calgary, June 1976, pp. 72-86.

1.17

Post-Tensioning Institute. "Chapter 2-Post-Tensioning Systems," *Post-Tensioning Manual*, 5th Edition, Phoenix, AZ, 1990, pp. 51-206.

1.18. "Section Properties for Concrete Masonry," *NCMA-TEK* 14-1, National Concrete Masonry Association, Herndon, VA, 1990.

1.19 He, L., and Priestley, M.J.N., Seismic Behavior of Flanged Masonry Shear Walls -Final Report, TCCMAR Report No. 4.1-2, November 1992, 279 pgs.

1.~~14~~~~15~~20. Dickey, W. and MacIntosh, A., "Results of Variation of b' or Effective Width in Flexure in Concrete Block Panels," Masonry Institute of America, Los Angeles, CA, 1971.

1.21. Arora, S.K. (1988). "Performance of masonry walls under concentrated load." Proceedings of the British Masonry Society, (2), 50-55.

1.22. Page, A.W., and Shrive, N.G. (1987). "Concentrated loads on hollow masonry – load dispersion through bond beams." The Masonry Society Journal, 6(2), T45-T51.

~~1.21~~1.23 Galambos, T. V., and Ellingwood, B. (1986). "Serviceability limit states: deflection." Journal of Structural Engineering, ASCE, 112(1), 67-84.

~~1.22~~1.24 Masonry Design for Buildings (Limit States Design). (1994). Canadian Standards Organization, S304.1-94.

~~1.23~~1.25 Branson, D.E., "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, 1965, pp.1-78.

~~1.24~~1.26 Horton, R.T., and Tadros, M.K. (1990). "Deflection of reinforced masonry members." ACI Structural Journal, 87(4), 453-463.

~~1.25~~1.27 Lee, R., Longworth, J., Warwaruk, J. (1983). "Behavior of restrained masonry beams." 3rd Canadian Masonry Symposium, Edmonton, Alberta, 37/1-16.

1.~~13~~14~~26~~8. ~~—~~. Pfister, J.F., "Influence of Ties on the Behavior of Reinforced Concrete Columns," ACI Journal, *Proceedings* V. 61, No. 5, American Concrete Institute, Detroit, MI, May 1964, pp. 521-537.

~~1.27~~1.29

~~1.15~~16 ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, Detroit, MI 1983, 111 pp.

1.~~28~~30~~17~~16~~5~~. Dickey, W.L., "Joint Reinforcement and Masonry," *Proceedings*, 2nd North American Masonry Conference, College Park, MD, Aug. 1982, The Masonry Society, Boulder, CO.

1.31 Brown, R.H. and Whitlock, A.R., "Strength of Anchor Bolts in Concrete Masonry," *Journal of the Structural Division*, American Society of Civil Engineers, New York, NY, Vol. 109, No. 6, June, 1983, pp. 1362-1374.

1.32 Allen, R., Borchelt, J. G., Klingner, R. E. and Zobel, R., "Proposed Provisions for Design of Anchorage to Masonry," *The Masonry Society Journal*, Vol. 18, No. 2, Dec. 2000, pp. 35-59.

~~1.29~~1.33~~18~~17~~6~~. Gulkan, P., Mayes, R.L., and Clough, R.W., "Shaking Table Study of Single-Story Masonry Houses Volumes 1 and 2," *Report* No. UCB/EERC-79/23 and 24, Earthquake Engineering Research Center, University of California, Berkeley, CA, Sept. 1979.

~~1.19~~187. ~~*"Autoclaved Aerated Concrete Properties, Testing and Design,"* RILEM Recommended Practice, RILEM Technical Committees 78-MCA and 51-ALC. Edited by: S. Aroni, G.J. de Grood, M.F. Robinson, G. Svanholm and F.H. Wittman, E & FN SPON, London, 1993.~~

~~1.20~~198. ~~—~~

~~1.22. Pfister, J.F., "Influence of Ties on the Behavior of Reinforced Concrete Columns," ACI JOURNAL, *Proceedings* V. 61, No. 5, American Concrete Institute, Detroit, MI, May 1964, pp. 521-537~~

~~1.20122~~

1.~~2~~1~~22~~33~~40~~ ASTM C 1093-95 (reapproved 2001), "Standard Practice for Accreditation of Testing Agencies for Unit Masonry," ASTM, West Conshohocken, Pennsylvania.

~~1.31. Arora, S.K. (1988). "Performance of masonry walls under concentrated load." Proceedings of the British Masonry Society, (2), 50-55.~~

~~1.32. Page, A.W., and Shrive, N.G. (1987). "Concentrated loads on hollow masonry – load dispersion through bond beams." The Masonry Society Journal, 6(2), T45-T51.~~

CHAPTER 2
ALLOWABLE STRESS DESIGN OF MASONRY

2.1 — General

2.1.2 *Load combinations*

The load combinations were selected by the committee and apply only if the legally adopted building code has none. Nine load combinations are to be considered and the structure must be designed to resist the maximum stresses resulting from the action of any load combination at any point of the structure. This Code requires that when simultaneous loading is routinely expected, as in the case of dead and live loads, the structure must be designed to fully resist the combined action of the loads prescribed by the legally adopted building code.

2.1.2.3 Previous editions of building codes have ~~customarily~~ used a higher allowable stress when considering wind or earthquake in a structure. Despite favorable historic performance, t~~T~~his increase has ~~come under attack~~been questioned, and there are different opinions ~~has been some confusion~~ as to the rationale for permitting the increase[2.1]. The committee ~~recognizes this situation but~~ has opted to continue to use the ~~increase~~ allowable stress~~es~~ increase in the traditional manner until documentation is available to warrant a change ~~(see Reference 2.1)~~.

2.1.3 *Design strength*

The structural adequacy of masonry construction requires that the compressive strength of masonry equal or exceed the specified strength. The specified compressive strength f'_m on which design is based for each part of the structure must be shown on the project drawings.

The 1995, 1999, 2002, and 2005 editions of the Code contained provisions to permit the use of ~~loading~~ strength-level load combinations in allowable stress design to compensate for the lack of service-level load combinations in previously referenced load standards. This procedure, which enabled the calculation of 'pseudo-strengths' on the basis of allowable stresses, is no longer included in the Code because recent editions of ASCE 7 include both service-level and strength-level load combinations. The user is directed to the 2005 edition of the Code for guidance on using strength-level load combinations whenever the general building code does not provide service-level load combinations.

2.1.3.4 *Strength requirements* — The strength of members and connections is based on working stress procedures modified by a factor. The nominal capacity is approximated as the allowable stress increased by $^{1}/_{3}$ (for the load combinations that include wind or earthquake in accordance with Section 2.1.2.3) and further multiplied by a factor of 2.5.

2.1.3.4.1 *Required strength* — For the initial version of Chapter 4, the response modification factor (R) and the deflection amplification factor (C_d) are taken the same as for unreinforced masonry. This requirement is intended to ensure that the structural response of prestressed masonry structures designed in accordance with these provisions essentially remains in the elastic range. When more experimental and field data are available on the ductility of both unbonded and bonded systems, R and C_d factors will be reviewed.

Only part of the reinforcement (nonprestressed) will eventually be replaced by bonded prestressing steel of equal cross sectional area. Unbonded prestressing steel may not be used to replace minimum reinforcement.

2.1.3.4.2 *Nominal strength* — The resulting nominal strength is approximately 3.3 times the allowable value obtained by using allowable stress design methodology. The design strength is equal to the nominal strength times the strength reduction factor, ϕ, to achieve a reliable design level value.

Because of the modifications of allowable stress values to strength design levels, some element strengths are calculated using steel stresses in excess of the specified yield. This procedure is correct, and produces designs that are intended to give similar levels of performance as using working stresses in combination with service level seismic loads.

2.1.4 *Anchor bolts solidly grouted in masonry*

2.1.4.1 *Test design requirements* — The design of anchor bolts is based on physical testing. Testing may be used to establish higher working loads than those calculated by Section 2.1.4.2. Many types of anchor bolts, such as expansion anchors, toggle bolts, sleeve anchors, etc., are not included in Section 2.1.4.2 and therefore, such anchors must be designed using test data. ASTM E 448 requires only three tests. The variation in test results for anchors embedded in masonry warrants an increase to the minimum of five stipulated. The variability of anchor bolt strength in masonry and the possibility that anchor bolts may be used in a nonredundant manner results in a safety factor of five.

2.1.4.2 *Plate, headed, and bent bar anchor bolts* These design values apply only to the specific bolts mentioned. They are readily available and are depicted in Fig.Figure 2.1-1.

2.1.4.2.1 The minimum embedment depth requirement is considered a practical minimum based on typical construction methods for embedding bolts in masonry. The validity of allowable shear and tension equations for small embedment depths, less than four bolt diameters, has not been verified by tests.

2.1.4.2.2 The results of tests on anchor bolts in tension showed that anchors failed by pullout of a conically shaped section of masonry, or by failure of the anchor itself. Bent bar anchor bolts (J-bolts) often failed by completely sliding out of the specimen. This was due to straightening of the bent end. Eq. (2-1) is the allowable tension load based on masonry failure. The area A_p is the projected area of the assumed failure cone. The cone originates at the bearing point of the embedment and radiates at 45º in the direction of the pull (See Fig.Figure 2.1-2). Comparisons of Eq. (2-1) to test results obtained by Brown and Whitlock[2.2] show an average factor of safety of approximately eight. Eq. (2-2) is the allowable load for anchor bolts based on failure of the bolt.

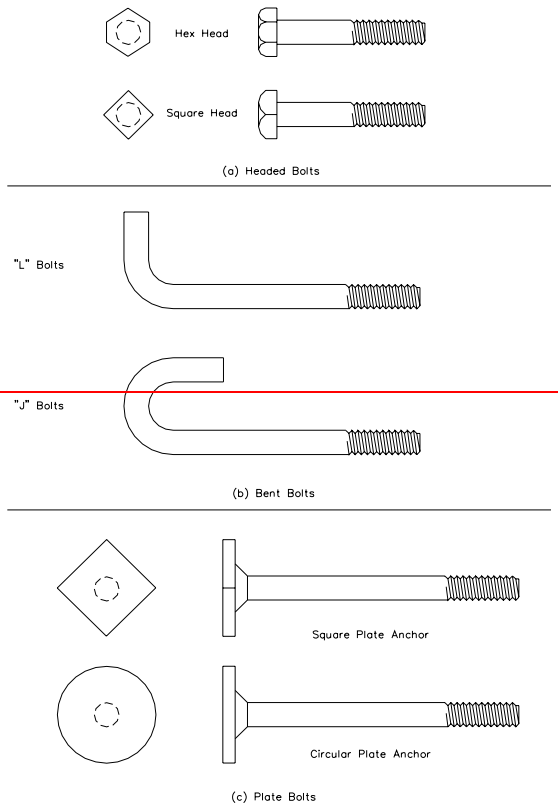
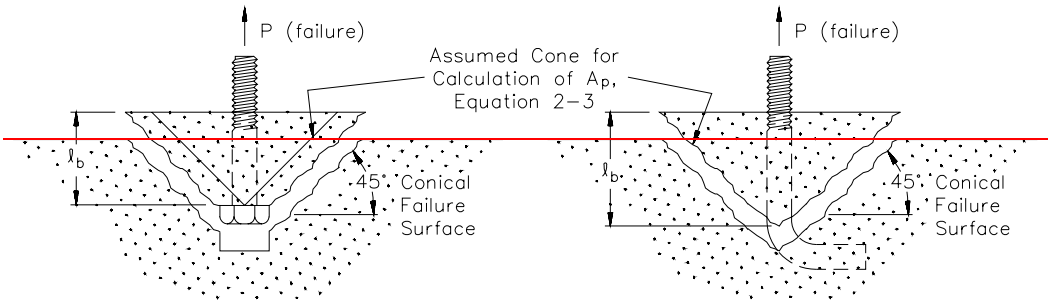


Fig.Figure 2.1-1 Anchor bolts



~~*Fig.Figure 2.1-2 Anchor bolts*~~

~~As anchor bolts are spaced closer together, the stresses within the masonry begin to become additive. Therefore, where the spacing between the anchors is less than $2l_b$, this Code requires that the projected areas used to calculate allowable load be reduced to reflect the additive stresses in the area of cone overlap as shown in Fig.Figure 2.1-4.~~

~~Test results[2.2] have shown that the pullout strength of bent bar anchors correlated best with a reduced embedment depth. This may be explained with reference to Fig.Figure 2.1-5. Due to the radius of the bend, stresses are concentrated at a point closer than the full embedment distance.~~

~~**2.1.4.2.3** Eq. (2-5) was derived from research done by Hatzinikolas et al.[2.3], and, when compared to tests done by Brown and Whitlock[2.2], the factors of safety range from approximately six to eight, respectively. Eq. (2-6) is based on the "shear friction" concept with a coefficient of friction equal to 0.6 and a safety factor of five. Fig.Figure 2.1-6 contains plots of Eq. (2-5) and (2-6).~~

~~Sufficient edge distances must be provided so that failures do not occur in modes that are not accounted for in the design equations.~~

~~(a) The reason is that with this amount of edge distance, a full failure cone can develop.~~

~~(b) The edge distance in the direction of the shear load was derived by equating the following expressions:~~

~~$V = \sqrt[4]{f'_m}(\pi m^2 / 2)$ (one-half stress cone directed toward free edge)~~

~~and~~

~~$V = 0.6(\pi D^2 / 4) f_y$ (anchor steel strength)~~

~~This resulted in the following expression:~~

$m = D\sqrt{(0.6/8)}(f_y / f'_m)$

For f_y = 60,000 psi (413.7 MPa) and f'_m = 1,000 psi (6.90 MPa), the required edge distance, m, equals 16.4D. (These equations are for inch-pound units only.)

2.1.4.2.4 *Combined shear and tension* — Test results[2.2] have shown that the strength of anchor bolts follows a circular interaction line. However, for simplicity and additional conservatism, this Code requires a straight line interaction between allowable shear and tension loads.

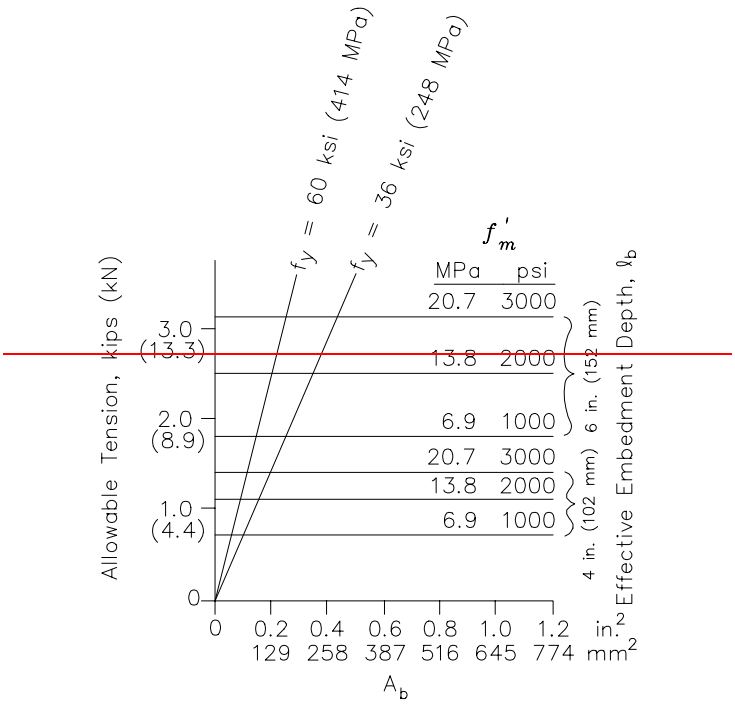
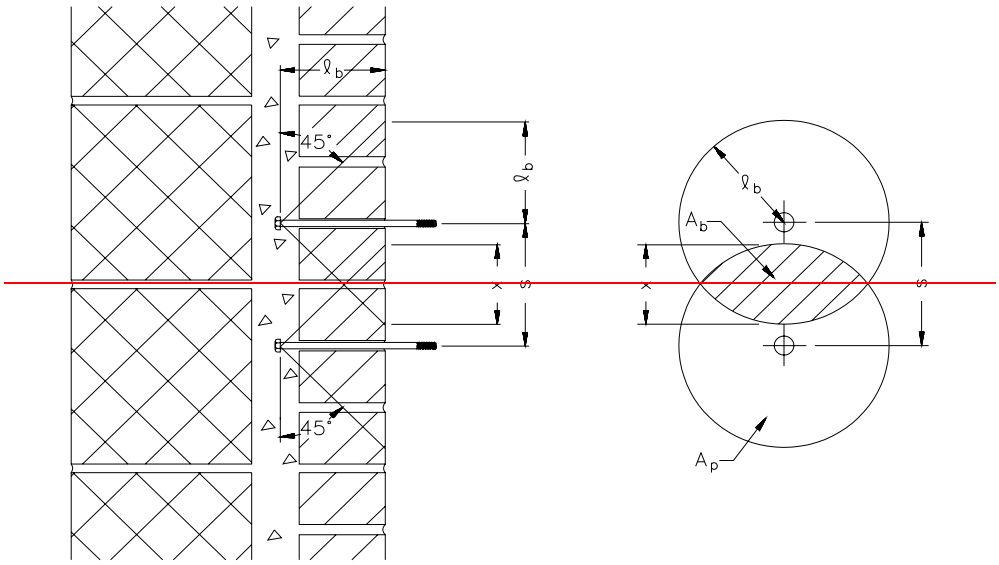
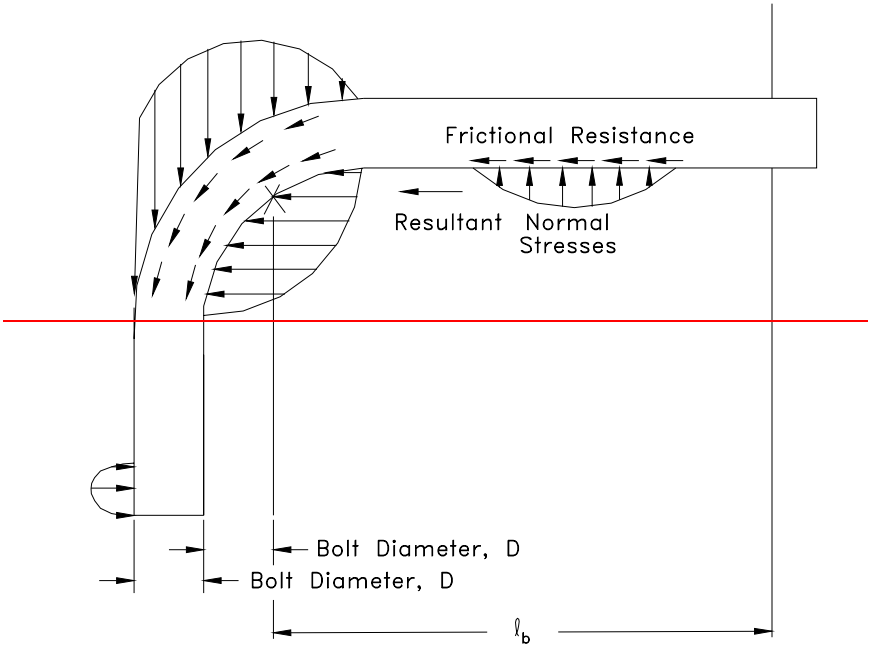


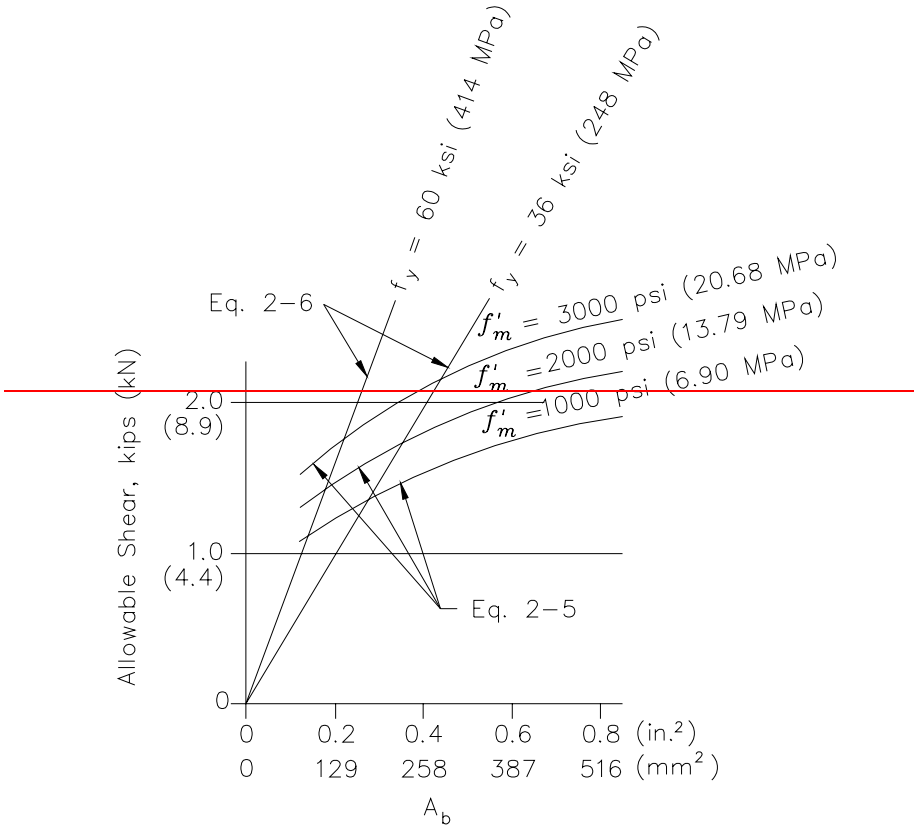
Fig.Figure2.1-3 — Allowable axial tension on anchor bolts



~~Fig.~~Figure ~~2.1-4 Anchor bolt cone area overlap~~



~~Fig.~~Figure ~~2.1-5 Stress distribution on bent anchor bars~~



~~*Fig.*~~*Figure 2.1-6 — Allowable shear stress on anchor bolts*

2.1.4 *Anchor bolts embedded in grout* - Allowable Stress Design anchor bolt provisions were obtained by calibrating corresponding Strength Design provisions to produce similar results. See Code Commentary 3.1.6.

2.1.5 *Multiwythe walls*

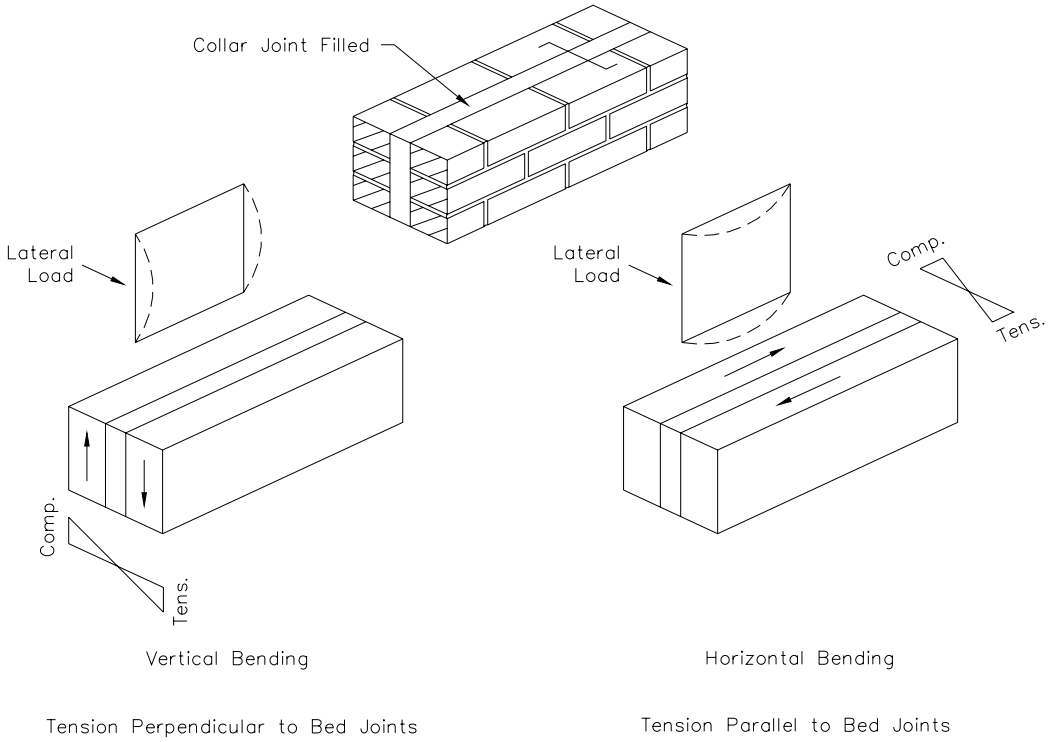
2.1.5.2 *Composite action* — Multiwythe walls act monolithically if sufficient shear transfer can occur across the interface between the wythes. See ~~Fig.~~Figure 2.1-7. Shear transfer is achieved with headers crossing the collar joint or with mortar- or grout-filled collar joints. When mortar- or grout-filled 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 stresses occurring at the interfaces are within the allowable limits prescribed.

Composite masonry walls generally consist of brick-to-brick, block-to-block, or brick-to-block wythes. The collar joint can be filled with mortar or grout, or the wythes can be connected with metal ties. 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 walls are particularly advantageous for resisting high loads, both in-plane and out-of-plane.

Limited test data[2.42, 2.53, 2.64] are available to document shear strength of collar joints in masonry.

Test results[2.42, 2.53] 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.[2.4-2] reported an average value of 52 psi (359 kPa) with a coefficient of variation of 21.6 percent. A low bound allowable shear value of 5 psi (34.5 kPa) 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[2.53]. Requirements for masonry headers (~~Fig.~~Figure 5.7-1) 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 allows up to 25 percent coring, is used. Headers do not provide as much ductility as metal tied wythes with filled collar joints. The influence of differential movement is especially critical when headers are used. The committee does not encourage the use of headers.



~~Fig.~~Figure 2.1-7 — Stress distribution in multiwythe walls of composite masonry

A strength analysis has been demonstrated by Porter and Wolde-Tinsae[2.~~7~~5, 2.~~8~~6] for composite 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[2.~~9~~7 - 2.~~12~~10]. Finite element models for analyzing the interlaminar shearing stresses in collar joints of composite walls have been investigated by Anand et al.[2.11~~3~~ - 2.16~~4~~]. 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 is controlled by the global strength of the wall, providing that the wythes are acting compositely.

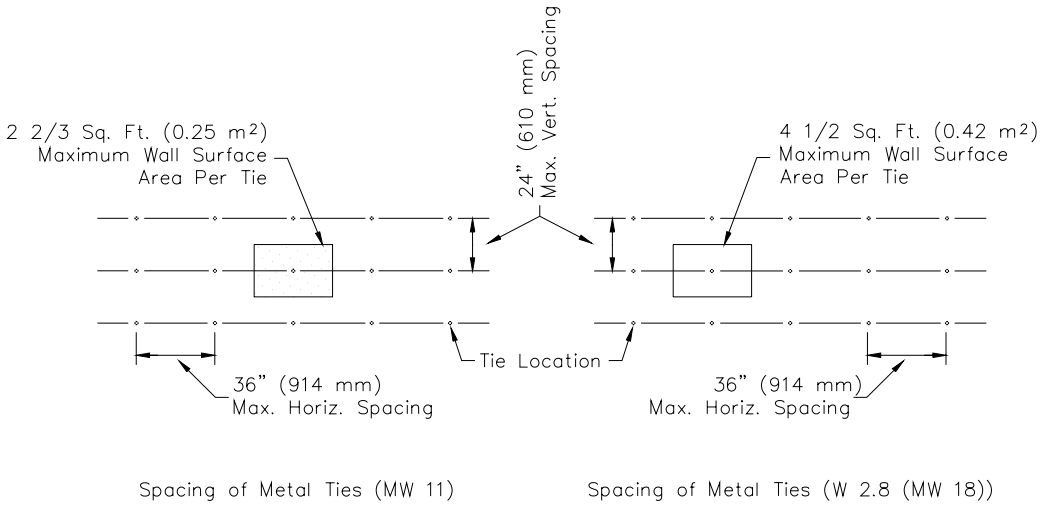
The size, number, and spacing of wall ties, shown in ~~Fig.~~Figure 2.1-8, has been determined from past experience. The limitation of Z-ties to walls of other than hollow units is also based on past experience.

2.1.5.3 *Noncomposite action* — Multiwythe walls may be constructed so that each wythe is separated from the others by a space that may be crossed only by ties. The ties force compatible

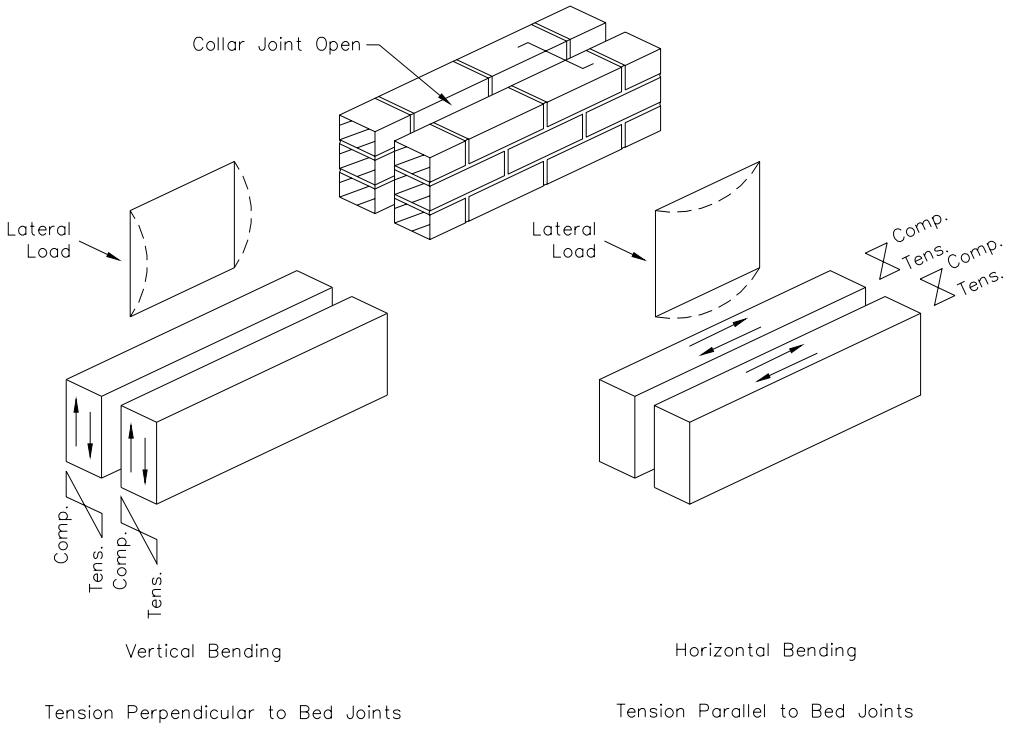
lateral deflection, but no composite action exists in the design. Weak axis bending moments caused by either gravity loads or lateral loads are assumed to be distributed to each wythe in proportion to its relative stiffness. See ~~Fig.~~ Figure 2.1-9 for stress distribution in noncomposite walls. Loads due to supported horizontal members are to be carried by the wythe closest to center of span as a result of the deflection of the horizontal member.

The size, number, and spacing of metal ties (~~Fig.~~ Figure 2.1-8) have been determined from past experience. 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 cavity wall ties with drips (bends in ties to prevent moisture migration) has been eliminated because of their reduced load capacity. In cavity walls, this Code limits the thickness of the cavity to 4½ in. (114 mm) to assure adequate performance. If cavity width exceeds 4½ in. (114 mm), the ties must be designed to carry the loads imposed upon them based on a rational analysis taking into account buckling, tension, pullout, and load distribution.

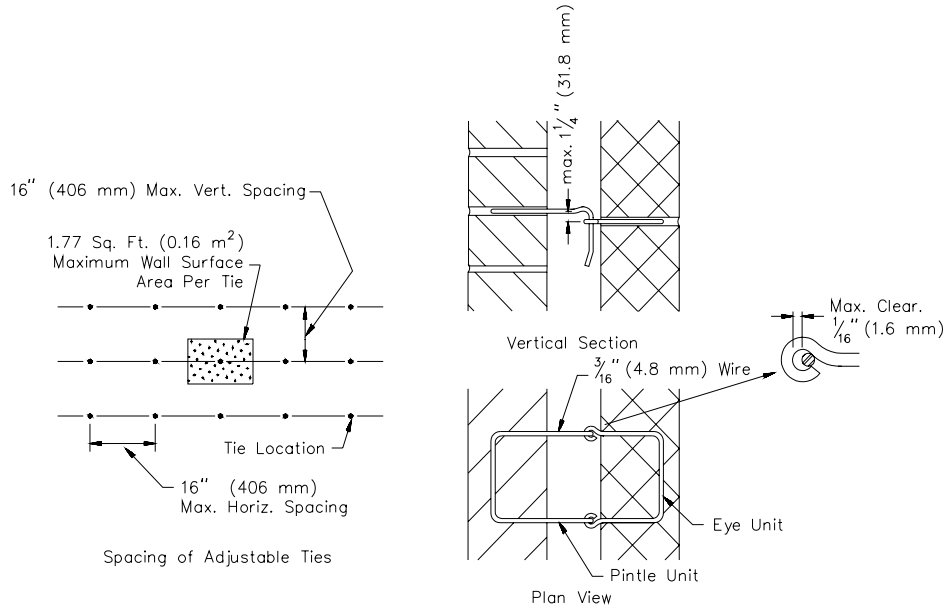
The NCMA[2.~~17~~ 15] and Canadian Standards Association (CSA)[2.~~18~~ 16] have recommendations for use in the design of ties for walls with wide cavities. The term cavity is used when the net thickness is 2 in. (51 mm) or greater. Two in. (51 mm) is considered the minimum space required for resistance to water penetration. A continuous air space of lesser thickness is referred to as a void (unfilled) collar joint. Requirements for adjustable ties are shown in ~~Fig.~~ Figure 2.1-10. They are based on the results in Reference 2.~~19~~17.



~~Fig.~~ Figure 2.1-8 — Wall tie spacing for multiwythe walls



~~Fig.~~ *Figure 2.1-9 — Stress distribution in multiwythe walls of noncomposite masonry*



~~Fig.~~ *Figure 2.1-10 — Adjustable ties*

2.1.6 *Columns*

~~2.1.6.1Columns 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 elements warrant the special requirements of this section that were selected after extensive committee consideration.~~

~~**2.1.6.1** The minimum nominal side dimension of 8 in. (203 mm) results from practical considerations.~~

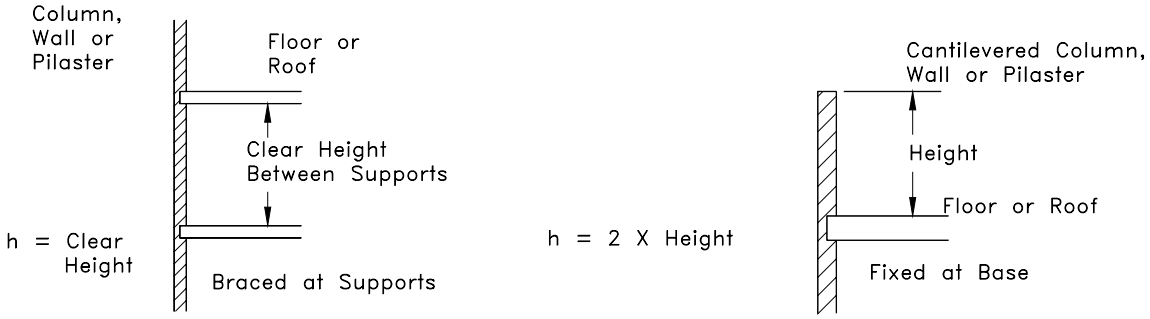
~~2~~ The limit of 25 for the effective height-to-least nominal dimension ratio is based on experience. Data are currently lacking to justify a larger ratio. See ~~Fig.~~Figure 2.1-11 for effective height determination.

2.1.6.3~~2~~ The minimum eccentricity of axial load (~~Fig.~~Figure 2.1-12) 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 (~~Fig.~~Figure 2.1-12).~~**2.1.6.4** Minimum vertical reinforcement is required in masonry columns to prevent brittle collapse. The maximum percentage limit in column vertical reinforcement was established based on the committee's experience. Four bars are required so ties can be used to provide a confined core of masonry.~~

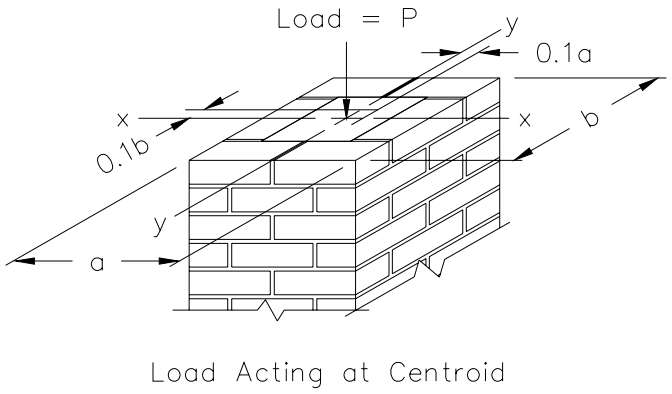
~~**2.1.6.5** *Lateral ties* Lateral reinforcement in columns performs two functions. It provides the required support to prevent buckling of longitudinal column reinforcing bars acting in compression and provides resistance to diagonal tension for columns acting in shear[2.20]. Ties may be located in the mortar joint.~~

~~The requirements of this Code are modeled on those for reinforced concrete columns. Except for permitting ¼ in. (6.4 mm) ties outside of Seismic Design Category D, E, or F, they reflect all applicable provisions of the reinforced concrete code.~~



If data (see Section 1.3) shows 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.

~~Fig.~~Figure 2.1-11 — Effective height, h, of column, wall, or pilaster



~~Fig.~~Figure 2.1-12 — Minimum design eccentricity in columns

2.1.7 *Pilasters*

Pilasters are masonry members that can serve one of several purposes. They may be visible, projecting from one or both sides of the wall, or hidden within the thickness of the wall as shown in ~~Fig.~~Figure 2.1-13. Pilasters aid in the lateral load resistance of masonry walls and may carry vertical loads.

2.1.8 *Load transfer at horizontal connections*

Masonry walls, pilasters, and columns may be connected to horizontal elements 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, 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.

~~**2.1.9** *Concentrated loads*~~

~~**2.1.9.1** Masonry laid in running bond is assumed to distribute the axial compressive stress resulting from a concentrated load along the length of wall as described in this Code. Stress is assumed to be transmitted across the head joints only of masonry laid in running bond. Thus, when other than running bond is used, concentrated loads can be assumed to be distributed across the length of only one unit unless a bond beam or other technique is used to distribute the load (Fig. 2.1-14).~~

~~**2.1.9.2** When the supporting masonry area is larger on all sides than the bearing area, this Code allows distribution of concentrated loads over a bearing area A_2 larger than A_1, determined as illustrated in Fig. 2.1-15. This is permissible because the confinement of the bearing area by surrounding masonry increases the bearing capacity of the wall in the vicinity of concentrated loads.~~

2.1.10 *Development of reinforcement embedded in grout*

2.1.10.1 *General* — 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.

~~All b~~Bars and longitudinal wires must be deformed.

2.1.10.2 *Development of wires in tension* — Eq. (2-~~8~~11) can be derived from the basic development length expression and an allowable bond stress u for deformed bars in grout of 160 psi (1103 kPa)[2.~~21~~18, 2.~~22~~19]. Research [2.2~~3~~0] has shown that epoxy-coated reinforcing bars require longer development length than uncoated reinforcing bars. The 50 percent increase in development length is consistent with the increase required in the ACI 318 provisions[1.14] for epoxy-coated bars and wires.

$l_d = d_b F_s / 4u = d_b F_s /4(160) = 0.0015 d_b F_s$

($l_d = 0.22 d_b F_s$ in SI units)

2.1.10.3 *Development of bars in tension or compression* — See the discussion in Code Commentary 3.3.3.4.

2.1.10.4 *Embedment of flexural reinforcement* — ~~Fig.~~Figure 2.1-~~16~~ 15 illustrates the embedment requirements of flexural reinforcement in a typical continuous beam. ~~Fig.~~Figure 2.1-~~17~~ 16 illustrates the embedment requirements in a typical continuous wall that is not part of the lateral load-resisting system.

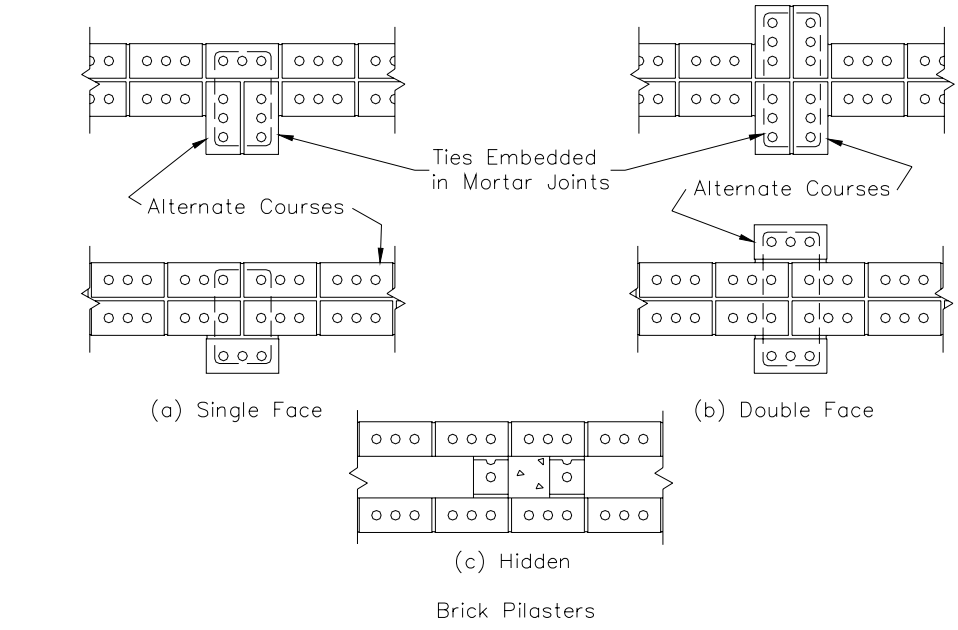
2.1.10.4.1.2 Critical sections for a typical continuous beam are indicated with a "c" or an "x" in ~~Fig.~~Figure 2.1-~~16~~15. Critical sections for a typical continuous wall are indicated with a "c" in ~~Fig.~~Figure 2.1-~~17~~16.

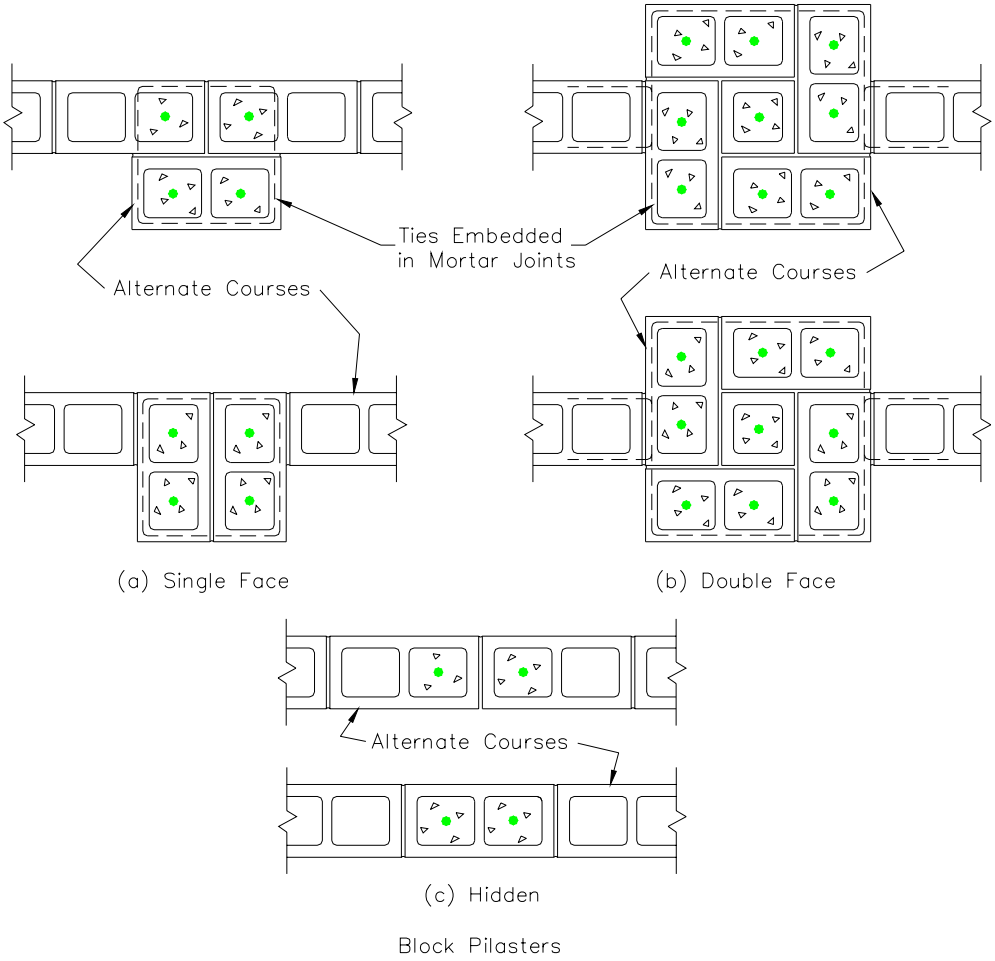
2.1.10.4.1.3 The moment diagrams customarily used in design are approximate. Some shifting of the location of maximum moments may occur due to changes in loading, settlement of supports, lateral loads, or other causes. A diagonal tension crack in a flexural member without stirrups may shift the location 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.

Cutoff points of bars to meet this requirement are illustrated in ~~Fig.~~Figure 2.1-~~16~~15.

When bars of different sizes are used, the extension should be in accordance with the diameter of bar being terminated. A bar 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 crosses the middepth of the member.





~~Fig.~~ Figure 2.1-13 — Typical ~~Pilasters~~ pilasters

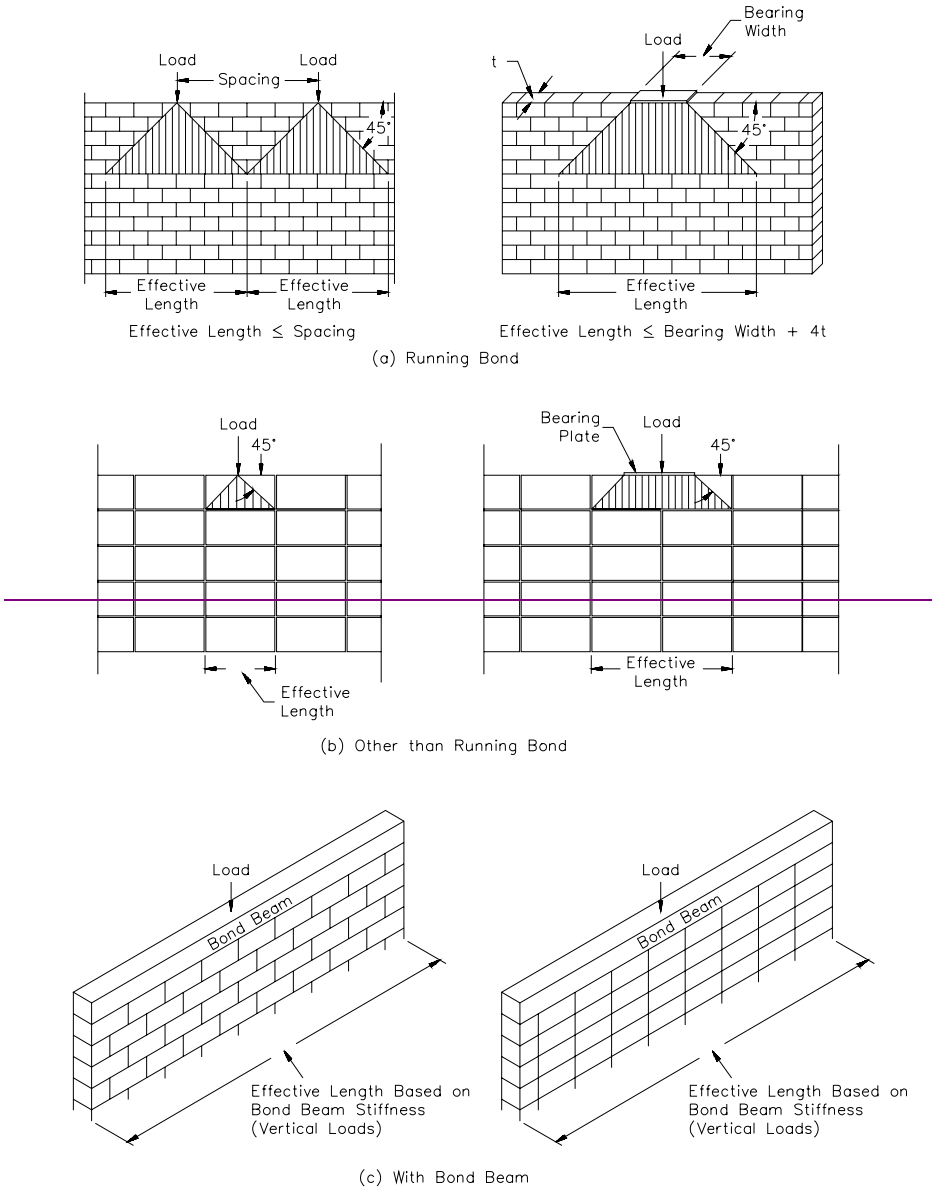


Fig. 2.1-14—Load distribution

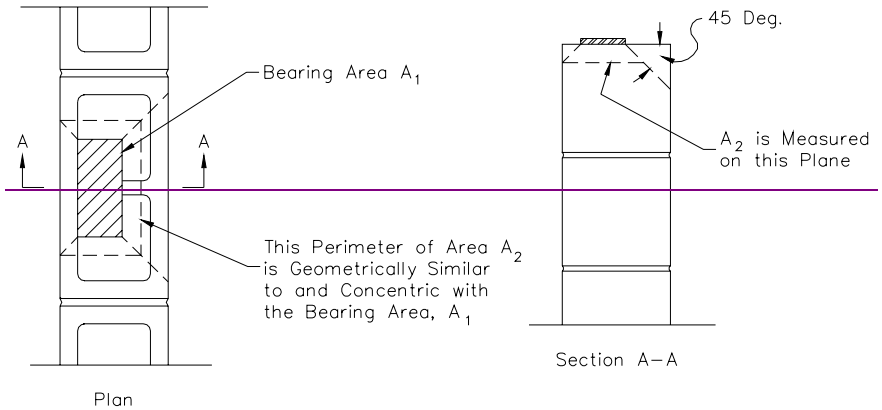


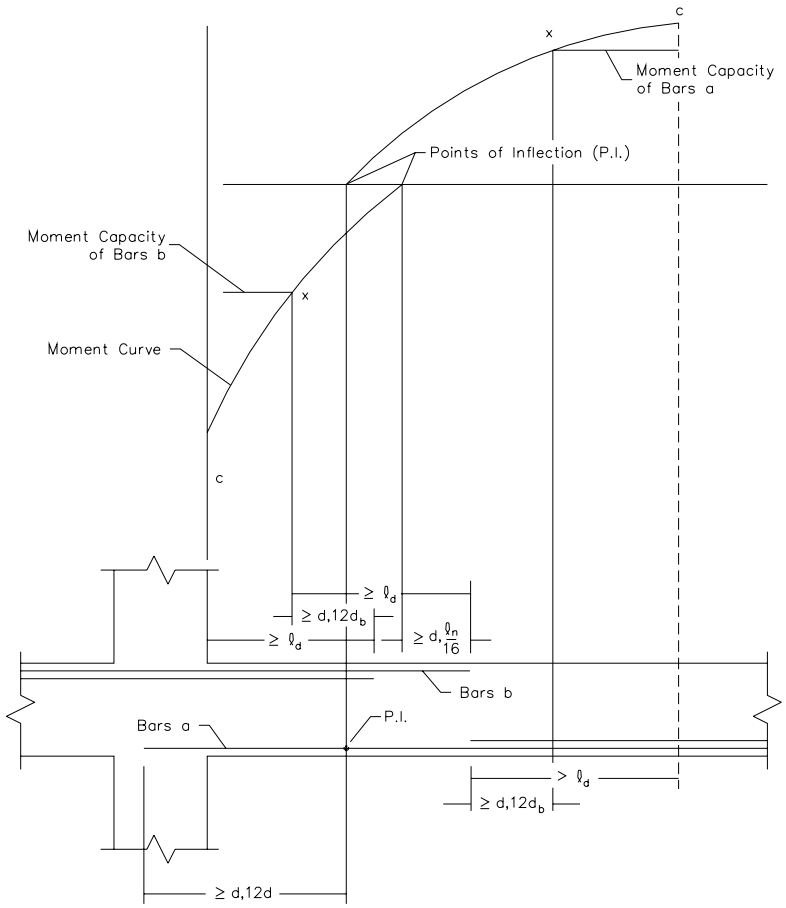
Fig. 2.1-15—Bearing areas

2.1.10.4.1.4 Peak stresses exist in the remaining bars wherever adjacent bars are cut off or bent in tension regions. In ~~Fig.~~Figure 2.1-~~16~~ 15 an "x" mark is used to indicate the peak stress points remaining in continuing bars after part of the bars have been cut off. If bars are cut off as short as the moment diagrams allow, these stresses become the full F_s, which requires a full embedment length as indicated. This extension may exceed the length required for flexure.

2.1.10.4.1.5 Evidence of reduced shear strength and loss of ductility when bars are cut off in a tension zone has been reported in Reference 2.~~24~~21. As a result, this 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.

2.1.10.4.1.6 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.

2.1.10.4.2 *Development of positive moment reinforcement* — When a flexural member is part of a primary lateral load-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 does not apply to excess reinforcement provided at the support.



~~Fig.~~ Figure 2.1-~~16~~ 15 — Development of flexural reinforcement in a typical continuous beam

2.1.10.4.3 *Development of negative moment reinforcement* — Negative reinforcement must be properly anchored beyond the support faces by extending the reinforcement l_d into the support. Other methods of anchoring include the use of a standard hook or suitable mechanical device.

Section 2.1.10.4.3.2 provides for possible shifting of the moment diagram at a point of inflection, as discussed under Commentary Section 2.1.10.4.1.3. This requirement may exceed that of Section 2.1.10.4.1.3 and the more restrictive governs.

2.1.10.5 *Hooks*

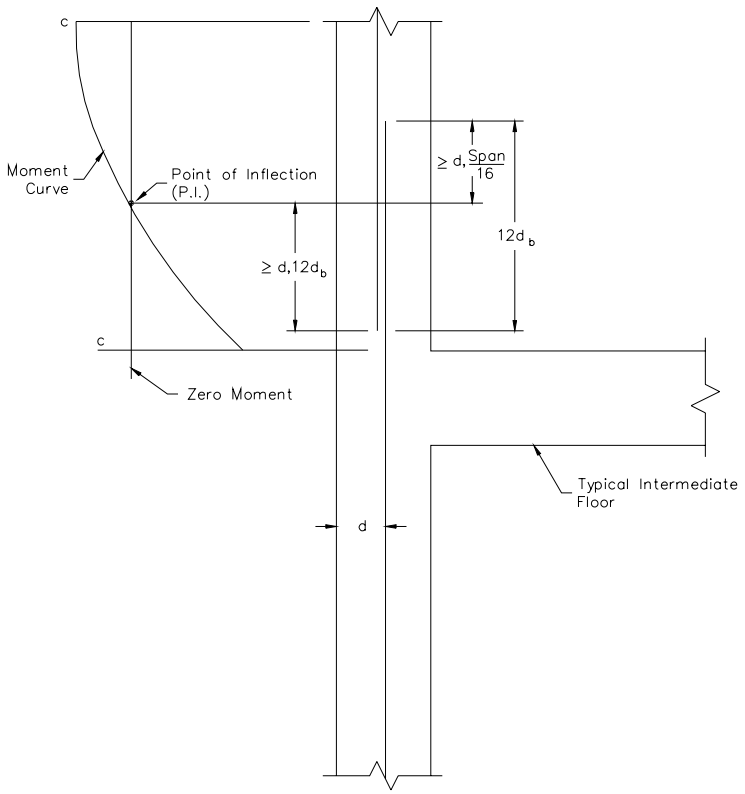
2.1.10.5.1 The allowable stress developed by a standard hook, 7,500 psi (51.7 MPa), is the accepted permissible value in masonry design. Substituting this value into Eq. (2-~~8~~11) yields the equivalent embedment length given. This value is less than half that given in Reference 1.14.

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

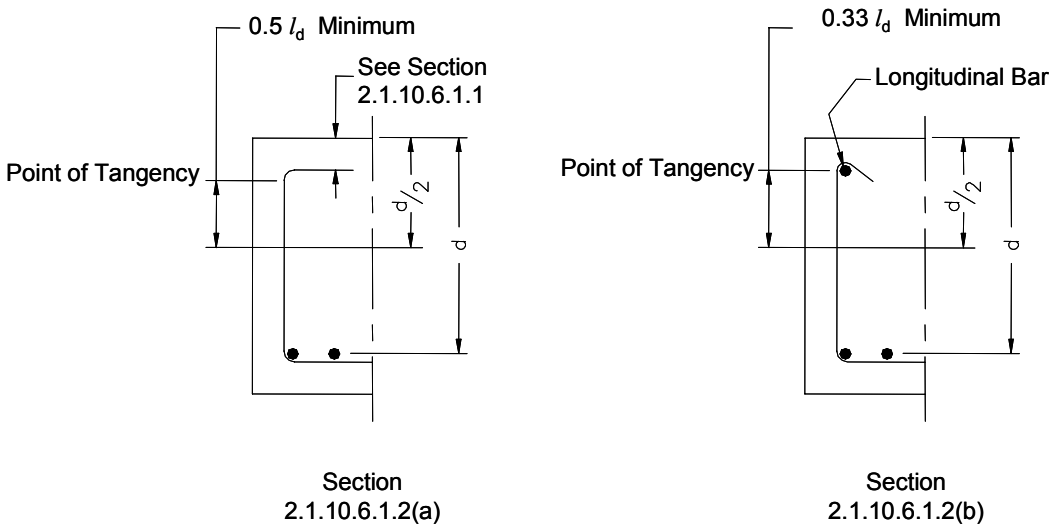
2.1.10.6 *Development of shear reinforcement*

2.1.10.6.1.1 Stirrups must be carried as close to the compression face of the member as possible because near ultimate load, flexural tension cracks penetrate deeply.

2.1.10.6.1.2 The requirements for anchorage of U-stirrups for deformed reinforcing bars and deformed wire are illustrated in ~~Fig.~~Figure 2.1-~~18~~17.



~~Fig.~~*Figure 2.1-~~17~~16 — Development of flexural reinforcement in a typical wall*



~~Fig.~~Figure 2.1-~~18~~17 — Anchorage of U-stirrups (deformed reinforcing bars and deformed wire)

2.1.10.6.1.2(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.

2.1.10.6.1.3 and 2.1.10.6.1.5 U-stirrups that enclose a longitudinal bar obviously have sufficient resistance in the tension zone of the masonry.

2.1.10.6.2 *Welded wire fabric* — Although not often used in masonry construction, welded wire fabric provides a convenient means of placing reinforcement in a filled collar joint. See Reference 2.2~~5~~2 for more information.

2.1.10.7 *Splices of reinforcement* — The importance of continuity in the reinforcement through proper splices is emphasized by the different requirements for the stress level to be transferred in the various types of splices[2.~~26~~23].

2.1.10.7.1.2 If individual bars in noncontact lap splices are too widely spaced, an unreinforced section is created, which forces a potential crack to follow a zigzag line. Lap splices may occur with the bars in adjacent grouted cells if the requirements of this section are met.

2.1.10.7.2 *Welded splices* — 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 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.

2.1.10.7.3 *Mechanical connections* — Full mechanical connections are also required to develop 125 percent of the yield strength in tension or compression as required, for the same reasons discussed for full welded splices.

2.1.10.7.4 *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.

2.2 — Unreinforced masonry

2.2.1 *Scope*

This section provides for the design of masonry members in which tensile stresses, not exceeding allowable limits, are resisted by the masonry. This has previously been referred to as unreinforced or plain masonry. Flexural tensile stresses may result from bending moments, from eccentric vertical loads, or from lateral loads.

A fundamental premise is that under the effects of design loads, masonry remains uncracked. One must be aware, however, that 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.

2.2.2 *Stresses in reinforcement*

Reinforcement may be placed in masonry walls to control the effects of movements from temperature changes or shrinkage.

2.2.3 *Axial compression and flexure*

2.2.3.1 For a member solely subjected to axial load, the resulting compressive stress f_a should not exceed the allowable compressive stress F_a; in other words, f_a/F_a should not exceed 1. Similarly, in a member subjected solely to bending, the resulting compressive stress f_b in the extreme compression fiber should not exceed the allowable compressive stress F_b, or again, f_b/F_b should not exceed 1.

This Code requires that under combined axial and flexure loads, the sum of the quotients of the resulting compression stresses to the allowable ($f_a/F_a + f_b/F_b$) does not exceed 1. This unity interaction equation is a simple portioning of the available allowable stresses to the applied loads, and is used to design masonry for compressive stresses. The unity formula can be extended when biaxial bending is present by replacing the bending stress quotients with the quotients of the calculated bending stress over the allowable bending stress for both axes.

In this 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 Eq. (2-~~10~~13) 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 Code Eq. (2-~~11~~14).
- For the practical range of *h/r* values, errors induced by ignoring the moment magnifier is relatively small, less than 15 percent.
- The overall safety factor of 4 included in the allowable stress equations is sufficiently large to allow this simplification in the design procedure.

The requirement of Eq. (2-~~11~~14) that the axial compressive load P not exceed $^1/_4$ of the buckling load P_e replaces the arbitrary upper limits on slenderness used in ACI 531[2.~~27~~24].

The purpose of Eq. (2-~~11~~14) 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 Eq. (2-~~15~~18), 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.

Eq. (2-~~11~~14) 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*, Eq. (2-~~13~~16) will overestimate the allowable compressive stress and Eq. (2-~~11~~14) may control.

The allowable stress values for F_a presented in Eqs. (2-~~12~~15) and (2-~~13~~16) are based on an analysis of the results of axial load tests performed on clay and concrete masonry elements. A fit of an empirical curve to this test data, ~~Fig.~~Figure 2.2-1, 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:

$$f'_m\left[1-(h/140r)^2\right] \qquad \text{(same with SI units)}$$

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, References 2.~~28~~25, 2.~~29~~26, and 2.~~30~~27 show that for a solid section, the critical compressive load for these members can be expressed by the formula

$$P_e = (\pi^2 E_m I_n / h^2)(1-2e/t)^3 \quad \text{(same with SI units)}$$

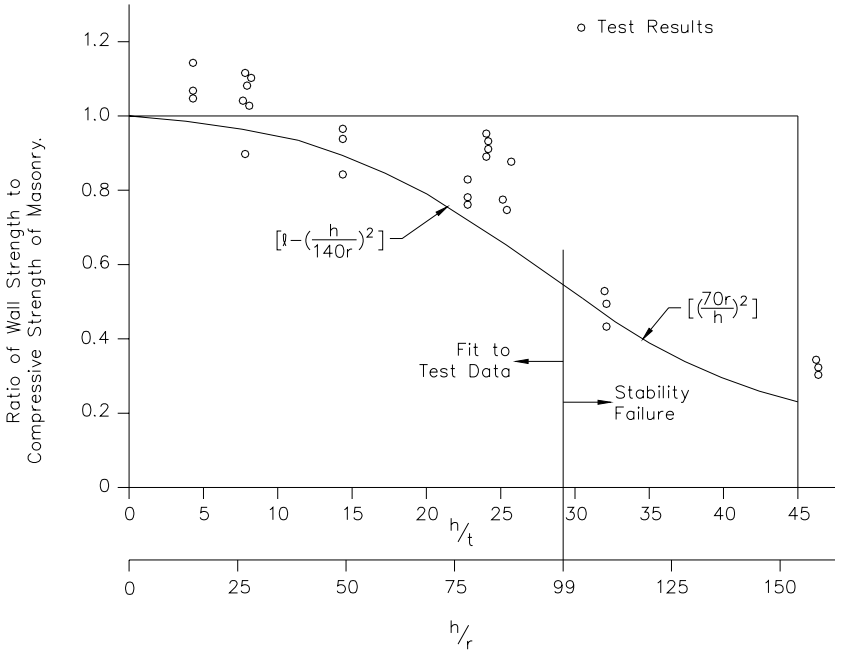
in which

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.



~~Fig.~~*Figure 2.2-1 — Slenderness effects on axial compressive strength*

For a solid rectangular section, $r = \sqrt{t^2/12}$. Making this substitution into the buckling load equation gives

$$P_e = \frac{\pi^2 E_m I_n}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3 \qquad (2\text{-}\sout{15}18)$$

Transforming the buckling equation using a minimum eccentricity of $0.1t$ (from Section 2.1.6.3) and an elastic modulus equal to $1000 f'_m$, the axial compressive stress at buckling failure amounts approximately to $[70(r/h)]^2 f'_m$. 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 [Eq. (2-~~13~~16)].

Flexure tests of masonry to failure have shown[2.31][28], 2.32[29], 2.33[30], 2.34 [31] that the compressive stress at failure computed by the straight line theory exceeds that of masonry failing under axial load. This phenomenon 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, computed 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:

$$f_b = \tfrac{4}{3} \times \left(\tfrac{1}{4}\right) f'_m = \left(\tfrac{1}{3}\right) f'_m$$

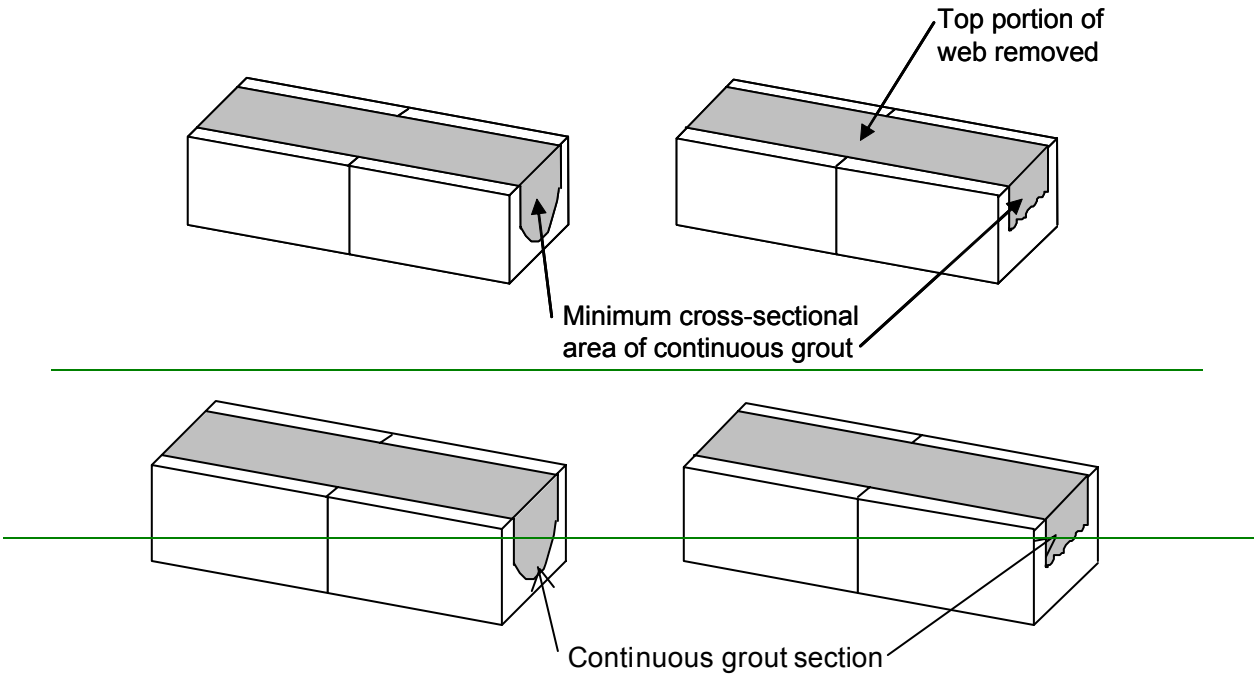
2.2.3.2 *Bending* — Allowable flexural tensile stresses for portland-cement lime mortar are traditional values.

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.[2.35][32], [2.36][33], 2.[37][34]

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 tensile stresses listed are for tension due to flexure under out-of-plane or in-plane loading. While it is recognized that in-plane and out-of-plane strain gradients are different, at these low stress levels this effect should be small. Flexural tensile stresses can be offset by axial compressive stress, but the resultant 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.

Stack bond masonry has historically 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 2.2-2.



~~Fig.~~Figure 2.2-2 — Continuous grout sections parallel to the bed joints

Test data using a bond wrench[2.3~~58~~, 2.3~~69~~] 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[2.3~~69~~, 2.3~~7480~~] 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[~~2.41~~2.38] 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.

Research work[~~2.42~~2.39] on flexural strength of concrete masonry has shown that grouting has a significant effect in increasing strength capacity 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 ~~all~~ 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[~~2.43~~2.40] has shown that flexural strength of unreinforced grouted concrete and clay masonry is largely independent of mortar type or cementitious materials.

For partial grouting, the footnote 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 25 psi (172 kPa), hence an allowable stress of 45 psi (310 kPa).

The presence of flashing and other conditions at the base of the wall can significantly reduce the flexural bond. The values in this Table apply only to the flexural tensile stresses developed between masonry units, mortar, and grout.

2.2.4 *Axial tension*

~~Tensile stresses~~Net axial tension in unreinforced masonry walls due to axially applied load are 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. ~~Cumulative c~~Compressive stress from dead load can be used to offset axial tension. ~~Masonry columns are required to have vertical reinforcement by Section 2.1.6.4.~~

2.2.5 *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) Stepped cracks form, alternating from head joint to bed joint.

In the absence of suitable research data, the committee recommends that the allowable shear stress values given in Code Section 2.2.5.2 be used for limiting out-of-plane shear stresses.

2.2.5.1 The theoretical parabolic stress distribution is used to calculate shear stress rather than the average stress. Many other codes use average shear stress so direct comparison of allowable values is not valid. Effective area requirements are given in Section 1.9.1. For rectangular sections, this equates to $^3/_2 \times V/A$. This equation is also used to calculate shear stresses for composite action.

2.2.5.2 Shear stress allowable values are applicable to shear walls without reinforcement. The values given are based on recent research[2.44–41 — 2.4744]. The 0.45 coefficient of friction, increased from 0.20, is shown in these tests. N_v is normally based on dead load.

2.3 — Reinforced masonry

2.3.1 *Scope*

The requirements covered 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. previously referred to as "reinforced masonry." The term reinforced masonry has been avoided to more accurately describe the conditions of design covered in Sections 2.2 and 2.3. Additionally, it will avoid confusion with masonry designed in accordance with the provisions of Section 2.2 in which the effect of joint and other reinforcement used in construction is neglected in the design.

Tension still develops in the masonry, but it is not considered to be effective in resisting design loads.

2.3.2 *Steel reinforcement - Allowable stresses* — These values have been in use for many years.

2.3.3 *Axial compression and flexure*

See Commentary for 2.2.3.1.

2.3.3.2 *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 2.3.3.2.1. Equation (2-1720) or (2-1821) controls the capacity of columns with large axial loads. The coefficient of 0.25 provides a factor of safety of about 4.0 against crushing of masonry. The coefficient of 0.65 was determined from tests of reinforced masonry columns and is taken from previous masonry codes[2.2724, 2.482.45]. 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 2.3.3.2.2. (See Commentary for Section 2.2.3.)

2.3.3.2.2 See Commentary for Section 2.2.3.1 for information on F_b.

The interaction equation used in Section 2.2.3 is not applicable for reinforced masonry and is therefore not included in Section 2.3.

~~**2.3.3.3** *Effective compressive width per bar* — The effective width of the compressive area for each reinforcing bar must be established. Fig. 2.3-1 depicts the limits for the conditions stated. Limited research[2.49] is available on this subject.~~

~~The limited ability of head joints to transfer stress when the masonry is laid in stack bond is recognized by the requirements for bond beams. Masonry units with open ends that are solidly grouted are assumed to transfer stress as indicated in Section 2.2.5.2(d), as for running bond.~~

~~The center-to-center bar 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.~~

~~**2.3.3.4**~~**2.3.3.3** *Beams* — The requirements for masonry members outlined are relatively straightforward and follow generally accepted engineering practice.

The minimum bearing length of 4 in. (102 mm) in the direction of span is considered a reasonable minimum for masonry beams over door and window openings to prevent concentrated compressive stresses at the edge of the opening. This requirement should also apply to beams and lintels in the plane of the wall.

2.3.3.4 *Walls* — The balanced reinforcement ratio for a masonry element designed by allowable stress design can be derived using standard engineering mechanics of a cracked, transformed section. The resulting equation is:

$$\rho_b = \frac{nF_b}{2F_s\left(n + \frac{F_s}{F_b}\right)}$$

~~W~~where ρ_b is the balanced reinforcement ratio resulting in a condition whereby both the reinforcement and masonry simultaneously reach their specified allowable stresses. Because the difference between specified design stresses and specified allowable stresses is not constant between reinforcement and masonry (F_s can range from 40% to 50% of f_y and F_b is taken equal to $1/3f'_m$), the committee agreed a more consistent application of this upper limit on the reinforcement ratio would be to replace the allowable stresses in the equation above with the corresponding specified stresses as shown in Code Equation 2-~~19~~22.

The equation is directly applicable for steel concentrated at the end of the shear wall. For distributed reinforcing, the reinforcement ratio is obtained as the total area of tension steel divided by *bd*.

~~To minimize lateral torsional buckling, Section 2.3.3.4.4~~2.3.3.3.2 ~~requires lateral bracing of the compression face in accordance with standard limits for beams of other materials. The requirement applies to simply supported beams as written. With continuous or fixed beams, the spacing may be increased.~~

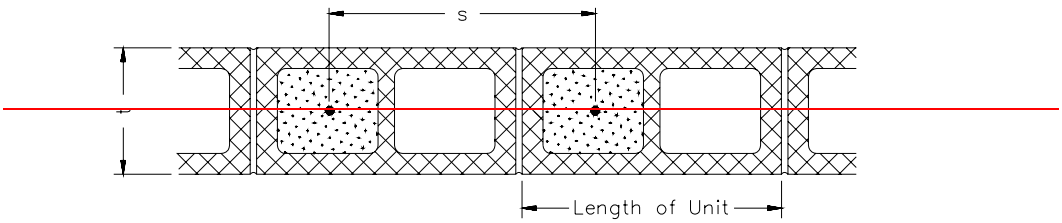
2.3.5 *Shear*

To compensate for a simplified method of analysis and unknowns in construction, the shear stresses allowed by this Code are conservative. When reinforcement is added to masonry, the shear resistance of the element is increased. Priestley and Bridgemen~~2.50~~2.46 concluded from a series of tests that shear reinforcement is effective in providing resistance only if it is designed to carry the full shear load. Thus, most codes do not add the shear resistance provided by the masonry to that provided by the steel. The amount of design shear reinforcement is specified to resist one hundred percent of the applied shearing load. See Commentary Section 2.2.5 and the flow chart for design of masonry members resisting shear shown in ~~Fig.~~Figure 2.3-2.

2.3.5.2 Eqs. (2-~~19~~2023) through (2-~~25~~2629) in Code Section 2.3.5.2 are derived from previous masonry codes~~2.27~~24, ~~2.51~~2.47, ~~2.52~~2.48.

2.3.5.2.1 Shear forces can act both vertically and horizontally under wind and seismic conditions in shear walls. Because the beams are designed as reinforced and are assumed to crack in flexure, the classical shear stress calculation used in Section 2.2 is replaced with an approximation of the maximum shear stress below the neutral axis. The approximation results from deleting the term "*j* " in the equation $f_v = V/bjd$.

2.3.5.2.3(a) The limits on the calculated shear stress in beams are in conformance with those given in previous masonry codes.



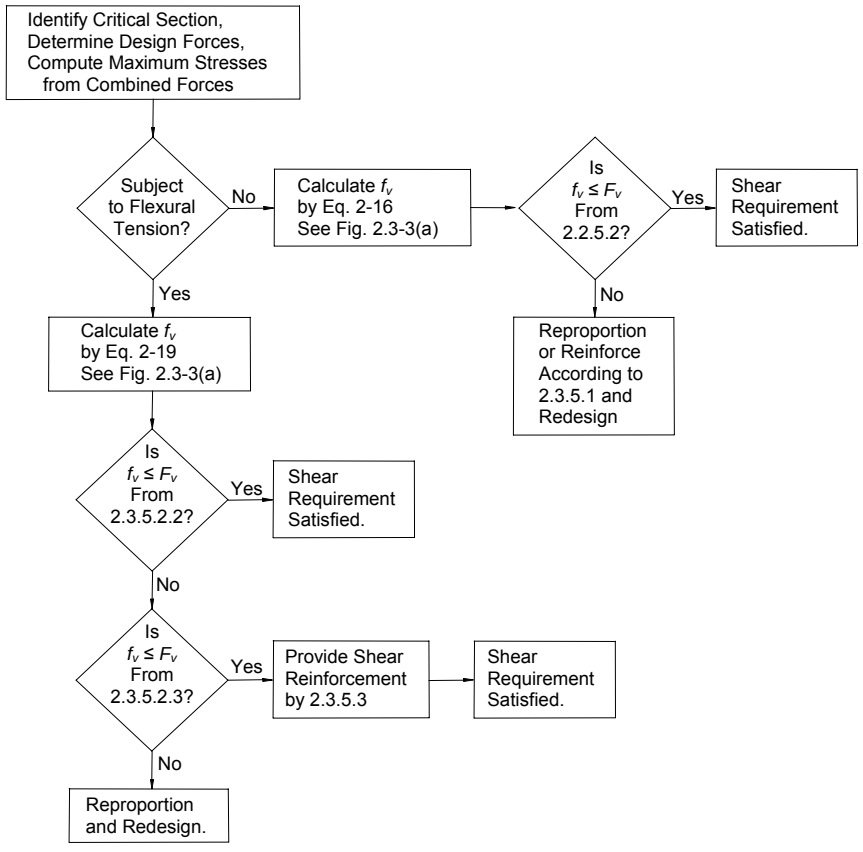
~~For masonry in other than running bond with bond beams spaced less than or equal to 48 in. (1219 mm) and running bond masonry, b equals the lesser of:~~

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

~~For masonry in other than running bond with bond beams spaced greater than 48 in. (1219 mm), b equals the lesser of:~~

~~b = s~~
~~b = length of unit~~

~~*Fig. 2.3-1 — Width of compression area*~~



~~Fig.~~*Figure 2.3-2 — Flow chart for shear design (PRIOR TO PUBLICATION: Staff to update figure again based on new Eq. Number (i.e., 2-19 is now 2-~~2023~~)*

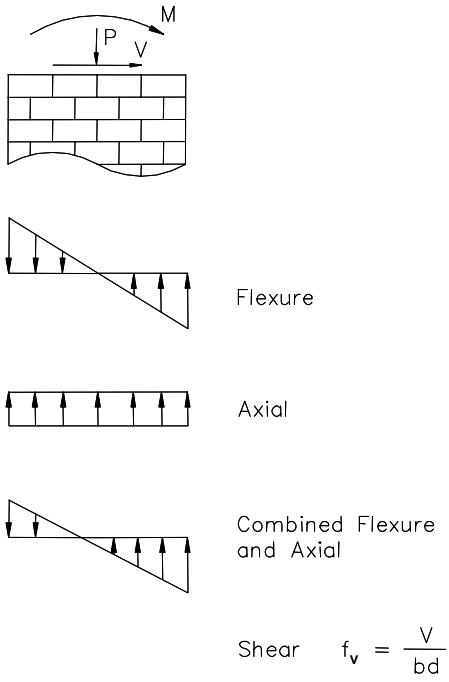
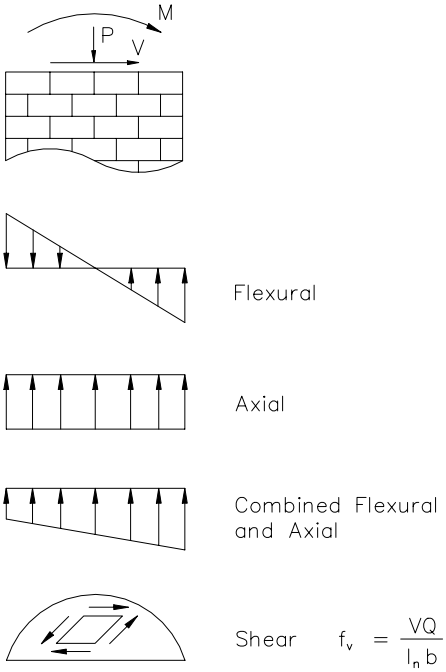


Fig 2.3-3(a) — Illustration of design section that is not subjected to tension



~~Fig.~~Figure 2.3-3(b) — Illustration of design section that is subjected to tension

2.3.5.3 Eq. (2-~~26~~27~~27~~30) may be derived by assuming a 45-degree shear crack extended from the extreme compression fiber to the centroid of the tension steel, which is the distance d. Forces are summed in the direction of the shear reinforcement and the doweling resistance of the longitudinal reinforcement is neglected. In Eq. (2-~~26~~27~~27~~30), for shear walls without shear reinforcement and for shear parallel to the plane of the wall, d_v may be substituted for d. Notice that for such shear walls, d_v may be either horizontal or vertical, depending on the direction of the shear and resulting reinforcement.

For shear walls, the longitudinal reinforcement is normally vertical and distributed along the length of the wall. The shear reinforcement is normally horizontal. In the development of the equation for shear walls, the 45-degree crack extends through more horizontal reinforcement than that obtained by using the depth to the centroid of the steel d. Thus, the use of d_v is justified. However, the designer must be cautioned that this is not always the case. For example, in a 10-ft (3.05-m) shear wall with vertical reinforcement located 2 ft (0.61 m) from each end (with no other vertical reinforcement), it would be unconservative to use d_v and the maximum reinforced length may be used in place of d_v.

2.3.5.3.1 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. The 48-in. (1219-mm) maximum spacing is an arbitrary choice that has been in codes for many years.

2.3.5.4 Shear across collar joints in composite masonry walls is transferred by the mortar or grout in the collar joint. Shear stress in the collar joint or at the interface between the wythe and the collar joint is limited to the allowable stresses in Section 2.1.5.2.2. Shear transfer by wall ties or other reinforcement across the collar joint is not considered.

2.3.5.5 The beam or wall loading within $d/2$ of the support is assumed to be carried in direct compression or tension to the support without increasing the shear load, provided no concentrated load occurs within the $d/2$ distance.

References

2.1.Ellifrit, D.S., "The Mysterious $^{1}/_{3}$ Stress Increase," *Engineering Journal*, ASIC, 4th Quarter, 1977.

~~2.2. Brown, R.H. and Whitlock, A.R., "Strength of Anchor Bolts in Concrete Masonry," *Journal of the Structural Division*, American Society of Civil Engineers, New York, NY, Vol. 109, No. 6, June, 1983, pp. 1362-1374.~~

~~2.3. Hatzinikolos, M., Longworth, J., and Warwaruk, J., "Strength and Behavior of Anchor Bolts Embedded in Concrete Masonry," *Proceedings*, 2nd Canadian Masonry Conference, Carleton University, Ottawa, Ontario, June, 1980. pp. 549-563.~~

2.42. McCarthy, J.A., Brown, R.H., and Cousins, T.E., "An Experimental Study of the Shear Strength of Collar Joints in Grouted and Slushed Composite Masonry Walls," *Proceedings*, 3rd North American Masonry Conference, Arlington, TX, June 1985, pp. 39-1 through 39-16, The Masonry Society, Boulder, CO.

2.53. Williams, R. and Geschwinder, L., "Shear Stress Across Collar Joints in Composite Masonry," presented at *Proceedings*, 2nd North American Masonry Conference, College Park, MD, 1982, *Paper* No. 8, The Masonry Society, Boulder, CO.

2.64. Colville, J., Matty, S.A., and Wolde-Tinsae, A.M., "Shear Capacity of Mortared Collar Joints," *Proceedings*, 4th North American Masonry Conference, Los Angeles, CA, Aug. 1987, V. 2 pp. 60-1 through 60-15, The Masonry Society, Boulder, CO.

2.75. Porter, M.L., Wolde-Tinsae, A.M., and Ahmed, M.H., "Strength Analysis of Composite Walls," *Advances in Analysis of Structural Masonry*, Proceedings of Structures Congress '86, American Society of Civil Engineers, New York, NY, 1986.

2.86. Porter, M.L., Wolde-Tinsae, A.M., and Ahmed, M.H., "Strength Design Method for Brick Composite Walls," *Proceedings*, 4th International Masonry Conference, London, Aug. 1987.

2.97. Wolde-Tinsae, A.M., Porter, M.L., and Ahmed, M.H., "Shear Strength of Composite Brick-to-Brick Panels," *Proceedings*, 3rd North American Masonry Conference, Arlington, TX, June 1985, pp. 40-1 through 40-13, The Masonry Society, Boulder, CO.

2.108. Wolde-Tinsae, A.M., Porter, M.L., and Ahmed, M.H., "Behavior of Composite Brick Walls," *Proceedings*, 7th International Brick Masonry Conference, Melbourne, New South Wales, Feb. 1985, V. 2, pp. 877-888.

2.~~11~~9. Ahmed, M.H., Porter, M.L., and Wolde-Tinsae, A.M., "Behavior of Reinforced Brick-to-Block Walls," Ph.D. dissertation, M. H. Ahmed, Iowa State University, Ames, IA, 1983, Part 2A.

2.1~~2~~0. Ahmed, M.H., Porter, M.L., and Wolde-Tinsae, A.M., "Behavior of Reinforced Brick-to-Block Walls," Ph.D. dissertation, M. H. Ahmed, Iowa State University, Ames, IA, 1983, Part 2B.

2.1~~3~~1. Anand, S.C. and Young, D.T., "A Finite Element Model for Composite Masonry," *Proceedings*, American Society of Civil Engineers, V. 108, ST12, New York, NY, Dec. 1982, pp. 2637-2651.

2.1~~4~~2. Anand, S.C., "Shear Stresses in Composite Masonry Walls," *New Analysis Techniques for Structural Masonry*, American Society of Civil Engineers, New York, NY, Sept. 1985, pp. 106-127.

2.1~~5~~3. Stevens, D.J. and Anand, S.C., "Shear Stresses in Composite Masonry Walls Using a 2-D Modes," *Proceedings*, 3rd North American Masonry Conference, Arlington, TX, June 1985, p. 41-1 through 40-15, The Masonry Society, Boulder, CO.

2.1~~6~~4. Anand, S.C. and Rahman, M.A., "Temperature and Creep Stresses in Composite Masonry Walls," *Advances in Analysis of Structural Masonry*, American Society of Civil Engineers, New York, NY, 1986, pp. 111-133.

2.1~~7~~5. "Anchors and Ties for Masonry," NCMA TEK 12-1, National Concrete Masonry Association, Herndon, VA, 1995, 6pp.

2.1~~8~~6. "Connectors for Masonry," (CAN 3-A370-M84), Canadian Standards Association, Rexdale, Ontario, 1984.

2.1~~9~~7. "Development of Adjustable Wall Ties," *ARF Project* No. B869, Illinois Institute of Technology, Chicago, IL, Mar. 1963.

~~2.20. Pfister, J.F., "Influence of Ties on the Behavior of Reinforced Concrete Columns," ACI JOURNAL, *Proceedings* V. 61, No. 5, American Concrete Institute, Detroit, MI, May 1964, pp. 521-537~~.

2.18~~21~~. Gallagher, E.F., "Bond Between Reinforcing Steel and Brick Masonry," *Brick and Clay Record*, V. 5, Cahners Publishing Co., Chicago, IL, Mar. 1935, pp. 86-87.

2.~~22~~19. Richart, F.E., "Bond Tests Between Steel and Mortar," Structural Clay Products Institute (Brick Institute of America), Reston, VA, 1949.

2.230. Treece, R.A., "Bond Strength of Epoxy-Coated Reinforcing Bars," Masters Thesis, Department of Civil Engineering, University of Texas at Austin, Austin, TX May, 1987.

2.241. Ferguson, P. M., and Matloob, F. N., "Effect of Bar Cutoff on Bond and Shear Strength of Reinforced Concrete Beams," ACI JOURNAL, *Proceedings* V. 56, No. 1, American Concrete Institute, Detroit, MI, July 1959, pp. 5-24.

2.252. Joint PCI/WRI Ad Hoc Committee on Welded Wire Fabric for Shear Reinforcement, "Welded Wire Fabric for Shear Reinforcement," *Journal*, Prestressed Concrete Institute, V. 25, No. 4, Chicago, IL, July-Aug. 1980, pp. 32-36.

2.263. ACI Committee 318, "Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, Detroit, MI, 1983, 155 pp.

2.274. ACI Committee 531, "Building Code Requirements for Concrete Masonry Structures (ACI 531-79) (Revised 1983)," American Concrete Institute, Detroit, MI, 1983, 20 pp.

2.285. Colville, J., "Simplified Design of Load Bearing Masonry Walls," *Proceedings*, 5th International Symposium on Loadbearing Brickwork, *Publication* No. 27, British Ceramic Society, London, Dec. 1978, pp. 2171- 2234.

2.296. Colville, J., "Stress Reduction Design Factors for Masonry Walls," *Proceedings*, American Society of Civil Engineers, V. 105, ST10, New York, NY, Oct. 1979, pp. 2035-2051.

2.3027. Yokel, F. Y., "Stability and Load Capacity of Members with no Tensile Strength," *Proceedings*, American Society of Civil Engineers, V. 97, ST7, New York, NY, July 1971, pp. 1913-1926.

2.3128. Hatzinikolas, M., Longworth, J., and Warwaruk, J., "Concrete Masonry Walls," *Structural Engineering Report* No. 70, Department of Civil Engineering, University of Alberta, Canada, Sept. 1978.

2.3229. Fattal, S.G. and Cattaneo, L.E., "Structural Performance of Masonry Walls Under Compression and Flexure," *Building Science Series* No. 73, National Bureau of Standards, Washington, DC, 1976, 57 pp.

2.330. Yokel, F. Y., and Dikkers, R. D., "Strength of Load-Bearing Masonry Walls," *Proceedings*, American Society of Engineers, V. 97, ST5, New York, NY, `May 1971, pp. 1593-1609.

2.341. Yokel, F. Y., and Dikkers, R. D., Closure to "Strength of Load-Bearing Masonry Walls," *Proceedings*, American Society of Engineers, V. 99, ST5, New York, NY, May 1973, pp. 948-950.

2.3~~5~~2. Melander, J.M. and Ghosh, S.K., "Development of Specifications for Mortar Cement," Masonry: Esthetics, Engineering and Economy, STP 1246, D. H. Taubert and J.T. Conway, Ed., American Society for Testing and Materials, Philadelphia, 1996.

2.3~~6~~3. Hedstrom, E.G., Tarhini, K.M., Thomas, R.D., Dubovoy, V.S., Klingner, R.E., and Cook, R.A., "Flexural Bond Strength of Concrete Masonry Prisms using Portland Cement and Hydrated Lime Mortars." The Masonry Society Journal, Vol. 9 No. 2, February 1991, Boulder, CO, pp. 8-23.

2.3~~7~~4. Borchelt, J.G. and J.A. Tann. "Bond Strength and Water Penetration of Low IRA Brick and Mortar," Proceedings of the Seventh North American Masonry Conference, 1996, South Bend, IN, pp. 206-216, The Masonry Society, Boulder, CO.

2.3~~8~~5. Brown, R. and Palm, B., "Flexural Strength of Brick Masonry Using the Bond Wrench," *Proceedings*, 2nd North American Masonry Conference, College Park, MD, Aug. 1982, The Masonry Society, Boulder, CO.

2.3~~9~~6. Hamid, A.A., "Bond Characteristics of Sand-Molded Brick Masonry," *The Masonry Society Journal*, V. 4, No. 1, Boulder, CO, Jan.-June 1985, pp. T-18,T-22.

2.4~~30~~7. Ribar, J., "Water Permeance of Masonry: A Laboratory Study," *Masonry: Properties and Performance*, STP-778, ASTM, Philadelphia, PA, 1982.

2.~~41~~38. Hamid, A.A., "Effect of Aspect Ratio of the Unit on the Flexural Tensile Strength of Brick Masonry," *The Masonry Society Journal*, Boulder, CO, V. 1, Jan.-June 1981.

2.~~42~~39. Drysdale, R.G. and Hamid, A.A., "Effect of Grouting on the Flexural Tensile Strength of Concrete Block Masonry," *The Masonry Society Journal*, V. 3, No. 2, Boulder, CO, July-Dec. 1984, pp. T-1,T-9.

2.4~~3~~0. Brown, R.H. and Melander, J.M., "Flexural Bond Strength of Unreinforced Grouted Masonry using PCL and MC Mortars," Proceedings of the 8th North American Masonry Conference, The Masonry Society, 1999.

2.4~~4~~1 Woodward, K. and Ranking, F., "Influence of Vertical Compressive Stress on Shear Resistance of Concrete Block Masonry Walls," U.S. Department of Commerce, National Bureau of Standards, Washington, D.C., Oct. 1984, 62 pp.

2.4~~2~~5. Pook, L.L., Stylianou, M.A., and Dawe, J.L., "Experimental Investigation of the Influence of Compression on the Shear Strength of Masonry Joints," *Proceedings*, 4th Canadian Masonry Symposium, Fredericton, New Brunswick, June 1986, pp. 1053-1062.

2.436. Nuss, L.K., Noland, J.L., and Chinn, J., "The Parameters Influencing Shear Strength Between Clay Masonry Units and Mortar," *Proceedings*, North American Masonry Conference, University of Colorado, Boulder, CO, Aug. 1978.

2.474. Hamid, A. A., Drysdale, R. G., and Heidebrecht, A. C., "Shear Strength of Concrete Masonry Joints," *Proceedings*, American Society of Civil Engineers, V. 105, ST7, New York, NY, July 1979, pp. 1227-1240.

2.485 "Recommended Practices for Engineered Brick Masonry," Brick Institute of America, Reston, VA, pp. 337, 1969.

2.496. ~~Dickey, W. and MacIntosh, A., "Results of Variation of *b'* or Effective Width in Flexure in Concrete Block Panels," Masonry Institute of America, Los Angeles, CA, 1971.~~

~~2.50.~~ Priestley, M.J.N., and Bridgeman, D.O., "Seismic Resistance of Brick Masonry Walls," *Bulletin*, New Zealand National Society for Earthquake Engineering (Wellington), V. 7, No. 4, Dec. 1974, pp. 167-187.

~~2.51~~2.47. "Specification for the Design and Construction of Load Bearing Concrete Masonry," (TR-75B), National Concrete Masonry Association, Herndon, VA, 1979.

~~2.52~~2.48. "Building Code Requirements for Engineered Brick Masonry," Brick Institute of America, Reston, VA, 1969, 36 pp.

CHAPTER 3
STRENGTH DESIGN OF MASONRY

~~**3.1.3** *Design strength*~~

~~**3.1.3.1** *Seismic design provisions* — 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 shear walls for lateral-load resistance, columns may also be incorporated for vertical capacity. The stipulation that lateral load resisting elements provide at least 80 percent of the lateral stiffness helps ensure that additional elements do not significantly contribute to the lateral stiffness. 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.~~

3.1.4 *Strength-reduction factors*

The strength-reduction factor incorporates the difference between the nominal strength provided in accordance with the provisions of Chapter 3 and the expected strength of the as-built masonry. The strength-reduction factor also accounts for the uncertainties in construction, material properties, calculated versus actual member strengths, as well as anticipated mode of failure.

3.1.4.1 *Combinations of flexure and axial load in reinforced masonry* — The same strength-reduction factor is used for the axial load and the flexural tension or compression induced by bending moment in reinforced masonry elements. The higher strength-reduction factor associated with reinforced elements (in comparison to unreinforced elements) reflects a decrease in the coefficient of variation of the measured strengths of reinforced elements when compared to similarly configured unreinforced elements.

3.1.4.2 *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 elements. The lower strength-reduction factor associated with unreinforced elements (in comparison to reinforced elements) reflects an increase in the coefficient of variation of the measured strengths of unreinforced elements when compared to similarly configured reinforced elements.

3.1.4.3 *Shear* — 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 3 require that shear capacity considerably exceed flexural capacity. Hence, the strength-reduction factor for shear is taken as 0.80, a value 33 percent larger than the historical value.

3.1.4.4 *Anchor bolts* — Because of the general similarity between the behavior of anchors embedded in grout and in concrete, and because available research data for anchors 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 anchors embedded in concrete.

3.1.4.5 *Bearing* — The value of the strength-reduction factor used in bearing assumes that some degradation has occurred within the masonry material.

3.1.5 *Deformation requirements*

3.1.5.1 ~~*Drift limits* — 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 (that is, the calculated displacement of one level relative to the level below caused by the effects of design seismic loads).~~

~~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 since it may give a false impression of the effects in critical stories. However, it is important when considering seismic separation requirements.~~

~~Overall or total drift is the lateral displacement of the top of a building relative to the base. The overall drift ratio is the 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. The maximum story drift ratio could exceed the overall drift ratio.~~

~~There are many reasons for controlling drift; one is to control member inelastic strain. Although the relationship between lateral drift and maximum strain is imprecise, so is the current state of knowledge of what strain limitations should be.~~

~~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.~~

~~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 drift limits, Δ_a, correspond to story drifts and, therefore, are applicable to each story (that is, 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 computing building separations, the distance should be sufficient to avoid damaging contact 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) should be followed.~~

~~**3.1.5.2**~~ *Deflection of unreinforced (plain) masonry* — The deflection calculations of unreinforced masonry are based on elastic performance of the masonry assemblage as outlined in the design criteria of Section 3.2.1.3.

3.1.5.~~3~~2 *Deflection of reinforced masonry* — Values of I_{eff} are typically about one-half of I_g for common solid grouted element configurations. Calculating a more accurate value using the cracked transformed section ~~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 resulting from unfactored loads as shown in the following equation. This equation has successfully been used for estimating the post-cracking flexural stiffness of both concrete and masonry[3.10].~~

~~$$I_{eff} = I_n\left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] \le I_n \le 0.5I_g$$~~

3.1.6 *Anchor bolts embedded in grout* - Design of anchor bolts embedded in grout may be based on physical testing or, for headed and bent-bar anchors, 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.

3.1.6.2 *Nominal strengths determined by test* – Many types of anchor bolts, such as expansion anchors, toggle bolts, sleeve anchors, etc., are not covered by Section 3.1.6.3 and, therefore, such anchors must be designed using test data. Testing may also be used to establish higher strengths than those calculated by Section 3.1.6.3. ASTM E448 requires only three tests. The variability of anchor bolt strength in masonry and the possibility that anchor bolts may be used in a non-redundant manner warrants an increase to the minimum of five tests stipulated by the Code. Assuming a normal distribution and a coefficient of variation of 20 percent for the test data, a fifth-percentile value for nominal strength is approximately obtained as 65 percent of the average strength value. Failure modes obtained from testing should be reported and appropriate ϕ factors used when establishing design strengths.

3.1.6.3 *Nominal strength determined by calculation for headed and bent-bar anchor bolts* – Design equations provided in the Code stem from research[3.1-3.7] conducted on headed anchor bolts and bent-bar anchors (J- or L-bolts) embedded in grout.

3.1.6.3.1.1 *Nominal axial tensile strength of headed anchor bolts* — Tensile strength of a headed anchor bolt is governed by yield and fracture of the anchor steel or 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. Steel strength is calculated using the effective tensile stress area of the anchor (that is, including the reduction in area of the anchor shank due to threads).

3.1.6.3.1.2 *Nominal axial tensile strength of bent-bar anchor bolts* – The tensile strength of a bent-bar anchor bolt (J- or L-bolt) is governed by yield and fracture of the anchor steel, by tensile cone breakout of the masonry, or by straightening and pullout of the anchor bolt from the masonry.

Capacities corresponding to the first two failure modes are calculated as for headed anchor bolts. Code equation (3-4) corresponds to anchor bolt pullout. The second term in equation (3-4) is the portion of the anchor bolt capacity due to bond between bolt and grout. Accordingly, Specification Article 3.2B requires that precautions be taken to ensure that the shanks of the bent-bar anchors are clean and free of debris that would otherwise interfere with the bond between anchor bolt and grout.

3.1.6.3.2 *Nominal shear strength of headed and bent-bar anchor bolts* - Shear strength of a headed or bent-bar anchor bolt is governed by yield and fracture of the anchor steel, by masonry crushing, or by masonry shear breakout. Steel strength is calculated using the effective tensile stress area (that is, threads are conservatively assumed to lie in the critical shear plane). Pryout (see Figure 1.16-5) is also a possible failure mode. The pryout equation (Equation 3-8) is adapted from ACI-318[3.8].

Under static shear loading, bent-bar anchor bolts do not exhibit straightening and pullout. Under reversed cyclic shear, however, available research[3.9] suggests that straightening and pullout may occur.

3.1.6.3.3 *Combined axial tension and shear* –Anchor bolts subjected to combined axial tension and shear must satisfy the linear interaction equation given by Equation 3-10.

~~**3.1.6** *Headed and bent-bar anchor bolts*~~

~~The design equations provided in the Code stem from research conducted on cast-in-place headed anchor bolts and bent-bar anchors (J- or L-bolts) in grout. Therefore, the application of these provisions to post-installed anchors may be in question. Due to the wide variation in configurations of post-installed anchors, designers are referred to product literature published by the manufacturers.~~

~~**3.1.6.1** *Nominal axial tensile strength of headed anchor bolts* — The tensile strength of a headed anchor bolt is governed by yield and fracture of the anchor steel or 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. (Refer to Commentary Fig.Figure 2.1-2.) Steel strength is calculated conventionally using the effective tensile stress area of the anchor (that is, including the reduction in area of the anchor shank due to threads).~~

~~**3.1.6.2** *Nominal axial tensile strength of bent-bar anchor bolts* — The tensile strength of a bent-bar anchor bolt (J- or L-bolt) is governed by yield and fracture of the anchor steel, by tensile~~

~~cone breakout of the masonry, or by straightening and pullout of the anchor from the masonry. Capacities corresponding to the first two failure modes are calculated as for headed anchor bolts. The Code requires that when the second term of Eq. (3-6) is used in the design calculations, precautions are taken to ensure that the shanks of the bent-bar anchors are clean and free of foreign debris that would otherwise interfere with the bond between the bolt and the grout.~~

~~**3.1.6.3** *Nominal shear strength of headed and bent-bar anchor bolts* — The shear strength of a headed or bent-bar anchor bolt is governed by yield and fracture of the anchor steel or by masonry shear breakout. Steel strength is calculated conventionally using the effective tensile stress area (that is, threads are conservatively assumed to lie in the critical shear plane). Under static shear loading, bent-bar anchor bolts (J- or L-bolts) do not exhibit straightening and pullout. Under reversed cyclic shear however, available research[3.1] suggests that straightening and pullout may occur.~~

3.1.7 *Nominal bearing strength*

~~Refer to~~ Commentary Section ~~2.1.9.2~~1.9.5 ~~for~~gives ~~provides~~ further information.

3.1.8 *Material properties*

~~Refer to~~ Commentary Section 1.8 provides ~~gives~~~~for~~ additional information.

3.1.8.1 *Compressive strength*

3.1.8.1.1 *Masonry compressive strength* — Design criteria are based on r~~R~~esearch[3.2~~10~~] ~~has been~~ conducted on structural masonry components having ~~a~~ compressive strengths from ~~in the range of~~ 1,500 to 6,000 psi (10.34 to 41.37 MPa). Design criteria are based on these research results. Design values therefore are limited to compressive strengths 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.

3.1.8.1.2 *Grout compressive strength* — Since most empirically derived design equations ~~relate the~~ calculate~~d~~ 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 ~~stipulated~~ an attempt to ensure that where the grout compressive strength may significantly control 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 ~~investigating for~~using higher material strengths ~~beyond this boundary~~.

3.1.8.2 *Masonry modulus of rupture*

~~**3.1.8.2.1** *Bending*~~ — The modulus of rupture values provided in Code Table ~~3.1.8.2.1~~3.1.8.2 are ~~intended to be directly related to~~ the allowable stress values for flexural tension multiplied by a factor of 2.5 to yield ~~an appropriate factor to approximate the~~ nominal strength values. While it is recognized that in-plane and out-of-plane strain gradients are different, at these low stress levels this effect should be small.

Stack bond masonry has historically 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 2.2-2.

The presence of flashing and other conditions at the base of the wall can significantly reduce the flexural bond. The values in this Table apply only to the flexural tensile stresses developed between masonry units, mortar, and grout.

3.1.8.3 *Reinforcement strength* — Research[3.~~2~~ 10] 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 is to minimize the over-strength unintentionally incorporated into a design.

3.2 — Unreinforced (plain) masonry

3.2.1.2 *Strength contribution from reinforcement* — Although reinforcement may still be present in unreinforced masonry, it is not considered in calculating design strength~~resisting design loads~~.

3.2.1.3 *Design criteria* — The design of unreinforced masonry requires that the structure performs elastically under ~~anticipated~~ design loads. The system response factors used in the design of unreinforced masonry assume an elastic response ~~of the structure~~.

3.2.2 *Flexure and axial strength of unreinforced (plain) masonry members*

3.2.2.2 *Nominal axial strength* — Commentary Section 3.3.4.1.1. gives additional information.

3.2.2.3 *Nominal axial strength ~~of unreinforced (plain) masonry members~~* — Refer to Commentary Section 3.3.4.1.1 for additional information.

3.2.2.4 *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:

$$\delta = \frac{C_m}{1 - \frac{P_u}{\phi_k P_{euler}}}$$

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[3.3~~11~~].

Euler's buckling load is obtained as $P_{euler} = \pi^2 E_m A_n r^2 / h^2$. Using E_m=700f'_m, which is the lower value of clay and concrete masonry, Euler's buckling load becomes:

$$P_{euler} = \frac{\pi^2 E_m A_n r^2}{h^2} = \frac{\pi^2 700 f'_m A_n r^2}{h^2} = A_n f'_m \left(\frac{83.1r}{h}\right)^2$$

Current design determines the axial capacity of walls with h/r>99 as $A_n f'_m (70r/h)^2$. Section 2.2.3.1 of the ~~e~~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 capacity of (70/83.1)^2=0.71 of Euler's buckling load

for clay masonry. The value of 0.71 is about 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 (70r/h) is used for Euler's buckling load instead of an explicit stiffness reduction factor. For ~~almost all~~ most walls, C_m = 1. The moment magnifier can thus be determined as:

$$\delta = \frac{1}{1 - \dfrac{P_u}{A_n f'_m \left(\dfrac{70r}{h}\right)^2}}$$

Figure 3.2-1 shows the ratio of the second-order stress, $\frac{P_u}{A_n} + \frac{\delta M_u}{S_n}$, divided by the first-order stress, $\frac{P_u}{A_n} + \frac{M_u}{S_n}$, when the second-order stress is at the strength design limit $\phi(0.8 f'_m)$. Typically slenderness effects are ignored if they contribute less than 5%[3.124]. From Figure 3.2-1, slenderness effects contribute less than 5% 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% 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%. Tall walls are those with *h/r* > 60 and must be designed using the moment magnifier approach.

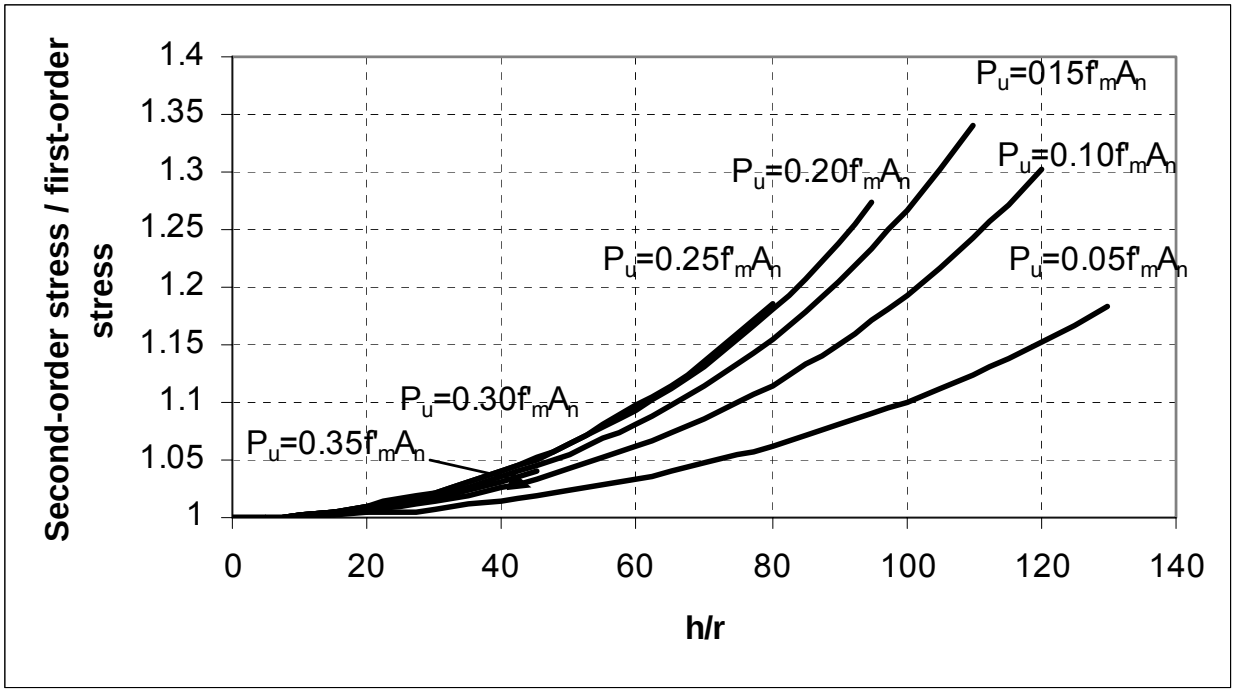


Figure 3.2-1 Ratio of second-order stress to first-order stress

3.2.3 *Axial tension*

Commentary Section 2.2.4 provides further information.

3.3 — Reinforced masonry

3.3.1 *Scope*

Reinforcement complements the high compressive strength of masonry with high tensile strength. Increased load-carrying capacity and greater ductility result from the use of reinforcement in the design of masonry structures.

3.3.2 *Design assumptions*

The design principles listed are those that traditionally have been used for reinforced masonry members.

The values for the maximum usable strain are based on research[3.3~~13~~5,3.~~4~~146] on masonry materials. 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.

While tension may still develop in the masonry of a reinforced element, it is not considered effective in resisting design loads, but is considered to contribute to the overall stiffness of a masonry element.

3.3.3 *Reinforcement requirements and details*

3.3.3.1 *Reinforcing bar size limitations* — The limit of using a No. 9 (M #29) bar is motivated by the desire to use a larger number of smaller diameter bars to transfer stresses rather than a fewer number of larger diameter bars. Some research investigations[3.2] have concluded that in certain applications masonry reinforced with more uniformly distributed smaller diameter bars performs better than similarly configured masonry elements using fewer larger diameter bars. While not ~~all~~ every investigation~~s are~~ is conclusive, the Committee does agree that incorporating larger diameter reinforcement may dictate unreasonable cover distances or development lengths. The limitations on clear spacing and percentage of cell area are indirect methods of preventing problems associated with over-reinforcing and grout consolidation. At sections containing lap splices, the maximum area of reinforcement should not exceed 8 percent of the cell area.

3.3.3.2 *Standard hooks* — Refer to Commentary Section ~~1.13~~1.15.5 for further information.

3.3.3.3 *Development* — The clear spacing between adjacent reinforcement does not apply to the reinforcing bars being spliced together. Refer to Commentary 3.3.3.4 for further information.

3.3.3.3.1 If individual bars in noncontact lap splices are too widely spaced, an unreinforced section is created, which forces a potential crack to follow a zigzag line. Lap splices may occur with the bars in adjacent grouted cells if the requirements of this section are met.

3.3.3.3.2.1 The edge vertical bar is the last reinforcing bar in walls without intersecting walls and is the bar at the intersection of walls that intersect. Hooking the horizontal reinforcement around a vertical bar located within the wall running parallel to the horizontal reinforcement would cause the reinforcement to protrude from the wall.

3.3.3.4 *Splices* — The required length of the lap splice is based on developing a minimum reinforcing steel stress of $1.25f_y$. This requirement provides adequate capacity 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 reinforcing steel and the surrounding grout. Testing[3.5-7-15, 3.16-6-8, 3.17-7-9, 3.8-10-18, 3.9-11-19] has shown that bond stress failure (or pull-out of the reinforcing steel) is only one possible mode of failure for lap splices. Other failure modes include rupture of the reinforcing steel and longitudinal splitting of masonry along the length of the lap. Experimental results of several independent research programs[3.15-5-7, 3.6-8-16, 3.7-9-17, 3.8-10-18, 3.9-11-19] were combined and analyzed to provide insight into predicting the necessary lap lengths for reinforcement splices in masonry construction.

To develop a reasonable design formula, multiple regression analysis was used to find the form of a good predictive model. The following equation yielded the best prediction of measured capacities of the tested splices[3.5-15-7]:

$$T_r = -176240 + 305.3 l_s + 252043 d_b^2 + 321.7\sqrt{f'_{mt}} + 3331.7 c_{cl}$$

Where:

T_r = predicted tensile strength of the splice, lb (N);

l_s = tested length of lap splice, in. (mm);

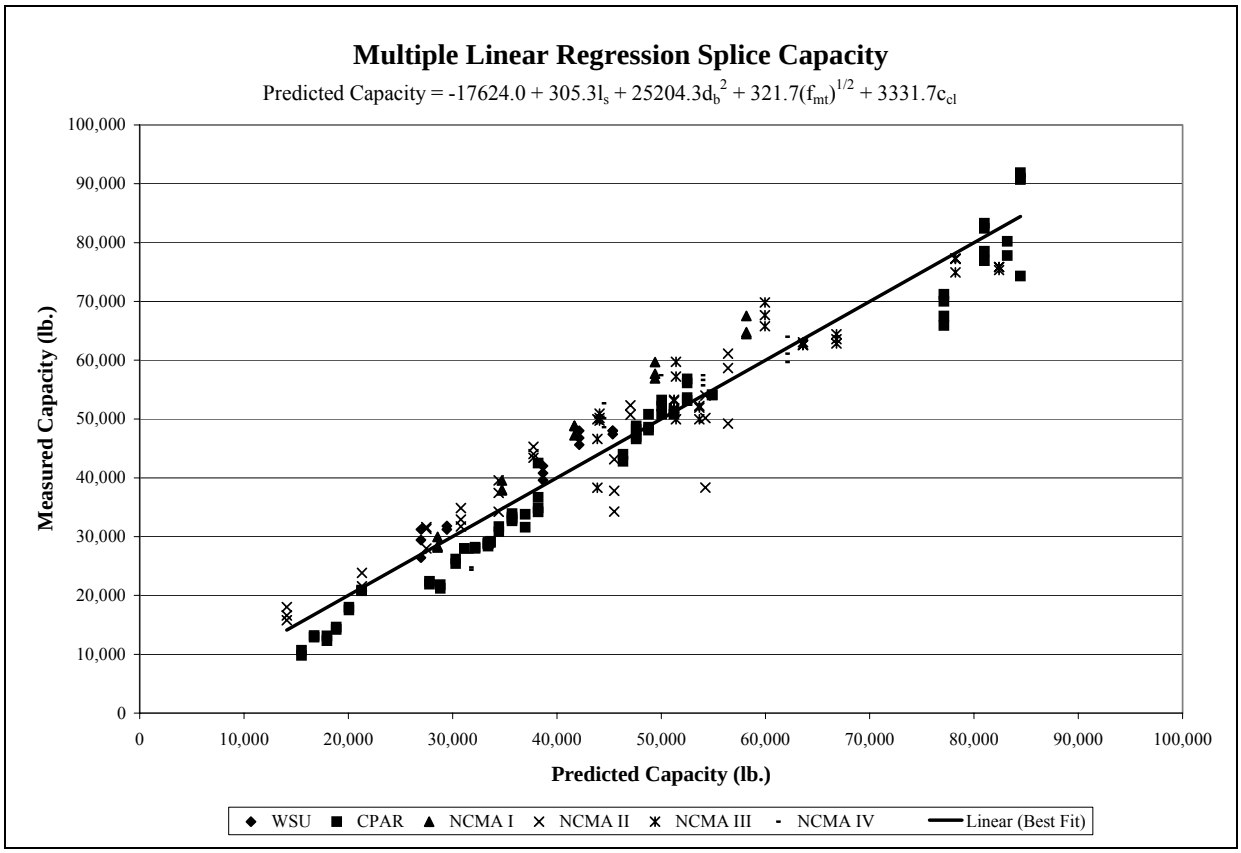
f'_{mt} = tested compressive strength of masonry, psi (MPa); and

c_{cl} = clear cover of structural reinforcement, in. (mm).

The square of the Pearson product moment correlation coefficient of this equation is 0.932, showing excellent correlation between the measured and predicted strength of the splices. Figure 3.3-1 graphically shows the equation predictions compared to results of the individual test programs.

Next, by replacing the predicted strength of the splice with $1.25A_b f_y$ (imposing the same requirement on lap splices as required for mechanical and welded splices) and solving for the resulting splice length yields: $l_s = \dfrac{1.25A_b f_y + 17624.0 - 25204.3d_b^2 - 321.7\sqrt{f'_{mt}} - 3331.7c_{cl}}{305.3}$

Since the form of this equation is impractical for design applications, Code equation (~~3-15~~3-16~~7~~) was fitted to the equation shown above.



~~Fig.~~ Figure 3.3-1 — Relationship between measured and predicted splice capacities

3.3.3.5 *Maximum area of flexural tensile reinforcement* — Longitudinal reinforcement in flexural members 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.

For all masonry components that are part of the lateral force resisting system, maximum reinforcement is limited in accordance with a prescribed strain distribution based on a tensile strain equal to a factor times the yield strain for the reinforcing bar 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 reinforcing steel is determined by considering the prescribed strain distribution, determining the corresponding stress and force distribution, and using statics to sum axial forces. For example, consider a solidly grouted shear wall subjected to in-plane loads with uniformly distributed reinforcement. The strain distribution is shown Figure 3.3-2, where ε_y is the yield strain and α is a tension reinforcement strain factor (3 for intermediate reinforced shear walls, 4 for special reinforced shear walls, and 1.5 for other masonry elements). The masonry force, C_m, the steel tension force, T_s, and the steel compression force, C_s, are determined as:

$$C_m = 0.8 f'_m \left[0.8 \frac{\varepsilon_{mu}}{\left(\varepsilon_{mu} + \alpha\varepsilon_y\right)} d_v \right] b$$

$$T_s = f_y A_s \left(\frac{\alpha\varepsilon_y}{\varepsilon_{mu} + \alpha\varepsilon_y} \right) \left[\frac{\alpha\varepsilon_y - \varepsilon_y}{\alpha\varepsilon_y} + \left(\frac{1}{2}\right) \frac{\varepsilon_y}{\alpha\varepsilon_y} \right]$$

$$C_s = f_y A_s \left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y} \right) \left[\frac{\varepsilon_{mu} - \varepsilon_y}{\varepsilon_{mu}} + \left(\frac{1}{2}\right) \frac{\varepsilon_y}{\varepsilon_{mu}} \right]$$

By statics, $P = C_s + C_m - T_s$, where:

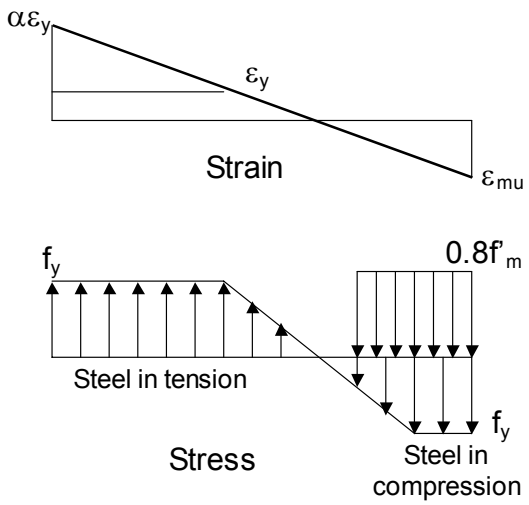
$$P = D + 0.75L + 0.525Q_E .$$

The maximum area of steel per unit length of wall is determined as:

$$\frac{A_s}{d_v} = \frac{0.64 f'_m \, b\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \frac{P}{d_v}}{f_y\left(\frac{\alpha\varepsilon_y - \varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right)}$$

For a solidly grouted member with just tension steel, the maximum reinforcement is:

$$\rho = \frac{A_s}{bd} = \frac{0.64 f'_m\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \frac{P}{bd}}{f_y}$$



~~Fig.~~*Figure 3.3-2 – Prescribed strain distribution and corresponding stress distribution.*

If there is compression steel with an area equal to the tension steel, A_s, the maximum reinforcement is:

$$\rho = \frac{A_s}{bd} = \frac{0.64 f'_m\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \frac{P}{bd}}{f_y - \min\left\{\varepsilon_{mu} - \frac{d'}{d}\left(\varepsilon_{mu} + \alpha\varepsilon_y\right), \varepsilon_y\right\} E_s}$$

where d' is the distance from the extreme compression fiber to the centroid of the compression reinforcement.

For partially grouted cross-sections subjected to out-of-plane loads, the maximum reinforcement is determined based on a solidly grouted member with tension steel only, provided that the neutral axis is in the flange. If the neutral axis is in the web, the maximum reinforcement is determined as:

$$\rho = \frac{A_s}{bd} \qquad \rho = \frac{A_s}{bd}$$

$$\rho = \frac{0.64 f'_m \left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y} \right) \left(\frac{b}{b_w} \right) + 0.80 f'_m \, t_{fs} \left(\frac{b - b_w}{bd} \right) - \frac{P}{bd}}{f_y} \qquad \rho = \frac{0.64 f'_m \left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y} \right) \left(\frac{b_w}{b} \right) + 0.80 f'_m \, t_{fs} \left(\frac{b - b_w}{bd} \right) - \frac{P}{bd}}{f_y}$$

where 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.

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. Thus, the capacity 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 7 Allowable Stress Load Combination 6 and consideration of the horizontal component of the seismic loading.

For structures expected to respond inelastically, the masonry compressive force is estimated using a rectangular stress block defined with parameters based on ~~recent~~ research carried out through~~done with~~ the Technical Coordinating Committee for Masonry Research (TCCMaR). For structures intended to undergo significant inelastic response, Sections 3.3.3.5.1, 3.3.3.5.2 and 3.3.3.5.3 are technically sound ways of achieving the design objective of inelastic deformation capacity. They are, however, unnecessarily restrictive for those structures not required to undergo significant inelastic deformation under the design earthquake and Section 3.3.3.5.4 addresses a relaxation of the maximum reinforcement limits.

For further discussion, see Reference 3.2, Report Nos. 3.1(a)-2, 3.1(c)-1, 3.1(c)-1, 4.1.-1, 4.1-2, and 9.2-4.

3.3.3.6 *Bundling of reinforcing bars* — This requirement stems from the lack of research on masonry with bundled bars.

3.3.4 *Design of beams, piers, and columns*

3.3.4.1 *Nominal strength*

3.3.4.1.1 *Nominal axial and flexural strength* — The nominal flexural strength of a member may be calculated using the assumption of an equivalent rectangular stress block ~~assumptions~~ as outlined in Section 3.3.2. ~~Refer to~~ Commentary Section 2.2.3 ~~for~~gives further information regarding slenderness effects on axial load capacity as taken into account with the use of Eq. (3-~~16~~178) and Eq. (3-~~17~~198). Eq. (3-~~16~~187) and Eq. (3-~~17~~198) apply to simply supported end conditions and transverse loading which results in a symmetric deflection (curvature) about the midheight of the element, if present. Where other support conditions or loading scenarios are known to exist, Eq. (3-~~16~~187) and Eq. (3-~~17~~198) should be modified accordingly to account for the effective height of the element or shape of the bending moment diagram over the clear span of the element. The weak-axis radius of gyration should be used in calculating slenderness-dependent reduction factors. The first coefficient, 0.80, in Eq. (3-~~16~~187) and Eq. (3-~~17~~198) accounts for unavoidable minimum eccentricity in the axial load.

3.3.4.1.2 *Nominal shear strength* — The limitations on maximum nominal shear strength are included to preclude critical (brittle) shear-related failures.

3.3.4.1.2.1 *Nominal masonry shear strength* — Eq. (3-~~21~~223) is empirically derived from research.[3.2]

3.3.4.1.2.3 *Nominal shear strength provided by reinforcement* — Eq. (3-~~22~~234) is empirically derived from research.[3.2]

3.3.4.2 *Beams* — This section applies to the design of lintels and beams.

3.3.4.2.2 *Longitudinal reinforcement*

3.3.4.2.2.1 Restricting the variation of bar sizes in a beam is included to increase the depth of the member compression zone and to increase member ductility. When incorporating two bars of significantly different sizes in a single beam, the larger bar requires a much higher load to reach yield strain, in effect “stiffening” the beam.

3.3.4.2.2.2 The requirement that the nominal flexural strength of a beam not be less than 1.3 ~~times~~ multiplied by the nominal cracking moment ~~strength~~ 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.

3.3.4.2.2.3 This exception provides sufficient additional reinforcement in members ~~where~~in which the amount of reinforcement required by Section 3.3.4.2.2.2 would be excessive.

3.3.4.2.3 *Transverse reinforcement* — Beams recognized in this section of the Code are often designed to resist only shear forces due to gravity loads. Flexural elements that are controlled by high seismic forces and lateral drift should be designed as ductile elements.

(a) Although some concerns have been raised regarding the difficulty in constructing beams containing a single bar stirrup, the Committee feels such spacing limitations within beams inhibits the construction of necessary lap lengths required for two-bar stirrups. Furthermore, the added volume of reinforcing steel 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 helps ensure 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 reinforcement requirements of this section establish limitations on the spacing and placement of steel in order to increase member ductility.

3.3.4.2.4 *Construction* — Although beams can physically be constructed of partially grouted masonry, the lack of research supporting the performance of partially grouted beams combined with the increased probability of brittle failure dictates this requirement.

3.3.4.2.5 *Dimensional limits*

~~(a) This limitation is judgment-based. The minimum spacing of lateral support for masonry beams is imposed to ensure lateral stability of the element.~~

~~(b)~~ Insufficient research has been conducted on beams of nominal depth less than 8 in. (203 mm).

3.3.4.3 *Piers*

3.3.4.3.1 Due to the less severe requirements imposed for the design of piers with respect to similar requirements for columns, the maximum axial force is arbitrarily limited to a relatively lower value.

3.3.4.3.2 *Longitudinal reinforcement* — These provisions are predominantly seismic related and are intended to provide the greatest ductility for the least expense. Pier elements not subject to in-plane stress reversals are not required to comply with this section.

3.3.4.3.3 *Dimensional limits* — Judgment-based dimensional limits are established for pier elements to distinguish their design from walls and to prevent local instability or buckling modes.

3.3.4.4 *Columns*

3.3.4.4.2~~43~~ *Dimensional limits* — These limitations are judgment-based. They are intended to prevent local instability or buckling modes.

3.3.5 *Wall design for out-of-plane loads*

~~3.3.5.3~~3.3.5.2 *Moment and deflection calculations* — The provisions of this section are derived from ~~test specimens that were modeled with pin connections top and bottom, thus having simple support conditions~~results of tests on simply supported specimens. ~~As a result,~~Because ~~the~~ maximum bending moment and deflection occur~~red at~~ near the mid-height of ~~the~~those specimens~~—hence~~, this section ~~only~~ includes only design equations ~~based on these observations~~for that condition. ~~In actual design and construction, there may be varying support conditions, thus changing~~ When actual conditions are not simple supports, the curvature of ~~the~~a wall under out-of-plane lateral loading will be different than that assumed by these equations. ~~Through proper calculation and u~~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 Section 3.3.5.

~~3.3.5.4~~3.3.5.3 *Walls with factored axial stress of 0.20 f'_m or less* — The criterion to limit vertical load on a cross section was included ~~since the test data used to establish~~because the slender wall design method was based on data from testing with typical roof loads. For slenderness ratios greater than 30, there is an additional limitation on the axial stress. There are currently no strength design provisions for axial stress greater than 0.20 f'_m.

The required moment due to lateral loads, eccentricity of axial load, and lateral deformations are 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 Eq. (3-25625~~25~~27).

The design formulas provide procedures for determining the nominal moment strength. These formulas take into account the effect of vertical loads increasing the capacity of the section.

~~3.3.5.5~~3.3.5.4 *Deflection design* — Historically, the recommendation has been to limit the service load deflection ~~under service load~~ to 0.01*h*. ~~However, t~~This deflection control ~~has also been viewed as neither a safety feature nor a safety requirement, but instead~~rather as an aesthetic and serviceability requirement considering intersecting walls and attachment to other materials. Hence, to address the concerns associated with safety, the deflection limitation of elements under service load was ~~reduced to~~ The committee has chosen a more stringent ~~a~~ value of 0.007*h*.

The Code limits the lateral deflection under service loads. ~~The wall will rebound to its normal vertical conditions.~~ 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.

Eq. (3-~~30~~312~~)~~) is for mid-height deflection for an uncracked section~~.~~, and Eq. (3-~~31~~332) is for mid-height deflection for a cracked section. ~~Assume that the wall deflects.~~ 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. ~~In these formulas, the effective area is used to calculate the moment of inertia, I_g. After the modulus of rupture is reached, the wall cracks and the moment of inertia is reduced. Thus, the cracked moment of inertia is assumed for the full height of the wall in calculating the additional deflection.~~ 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 solid grouted or partially grouted cross section is usually the same as that for a hollow section since the compression stress block is generally within the thickness of the face shell.

These formulas ~~are~~ represent good approximations to ~~the~~ test results ~~achieved in the tests~~, assuming that the wall is ~~pin-connected~~ simply supported top and bottom ~~in a simple flexural condition~~, and ~~the wall~~ 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 effective moment of inertia would be based upon the minimum solid net areas of solid or partially grouted sections.~~ The cracking moment, M_{cr}, is the calculated moment corresponding to

first cracking. The cracking moment was previously given in the Code as the section modulus ~~times~~ multiplied by the modulus of rupture. The Code has been changed so it is now permissible to include the applied axial force in the calculation of the cracking moment.

3.3.6 *Wall design for in-plane loads*

3.3.6.5 The maximum reinforcement requirements of Section 3.3.3.5 are intended to ensure that an intermediate or a special reinforced masonry shear wall has sufficient inelastic deformation capacity ~~would have a satisfactory degree of inelastic deformability~~ under the design-basis earthquake of ASCE 7 ~~02~~ 05 or the model building codes. Inelastic deformability is the ability of a structure or structural element 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 element to that excitation is elastic (that is, associated with no residual displacement or damage). In the alternative shear wall design approach given in Sections 3.3.6.6 through 3.3.6.10, such inelastic deformability is sought to be ensured by means of specially confined boundary elements, making it unnecessary to comply with the maximum reinforcement requirements. These requirements are therefore waived.

3.3.6.6 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 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 1990's, 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, showing that a reinforced concrete shear wall passing through the "screens" could not develop sufficiently high ~~enough~~ 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 3.3.2 (c).

3.3.6.7 Two approaches for evaluating detailing requirements at wall boundaries are included in Section 3.3.6.7. Section 3.3.6.8 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. This approach was first introduced in ACI 318-99 for the design of special reinforced concrete shear walls. The provisions of Section

3.3.6.9 are similar to those of 1995 and earlier editions of ACI 318 (retained in ACI 318-99 and 318-02), and have been included because they are conservative for assessing required transverse reinforcement at wall boundaries for many walls. The requirements of Section 3.3.6.10 apply to shear walls designed by either Section 3.3.6.8 or 3.3.6.9.

3.3.6.8 Section 3.3.6.8 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 ~~Fig.~~Figure 3.3-3, is adapted from a paper by Wallace[3.11~~12~~20], ~~in which he provided~~which provides background to the design provisions for special reinforced shear walls of ACI 318-99 (retained unchanged in ACI 318-~~02~~05). 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 ~~Fig.~~Figure 3.3-3. The ACI 318-~~99~~ provisions as well as the ~~proposed~~ provisions of this Code are based on a simplified version of the model presented in ~~Fig.~~Figure 3.3-3(a). The simplified model, shown in ~~Fig.~~Figure 3.3-3(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 ~~Fig.~~Figure 3.3-3, the relationship between the top displacement and the curvature at the base of the wall is:

$$C_d \delta_{ne} = \theta_p h_w = (\phi_u \ell_p) h_w = \left(\phi_u \frac{\ell_w}{2} \right) h_w \qquad (1)$$

assuming that $\ell_p = \ell_w / 2$, as is permitted to be assumed by the 1997 UBC,

where ϕ_u = ultimate curvature, and

θ_p = plastic rotation at the base of the wall.

If at the stage where the top deflection of the wall is δ_{ne}, 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 3.3-4 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 Eq. (1),

$$C_d \delta_{ne} = \left(\frac{\varepsilon_{mu}}{c_{cr}} \frac{\ell_w}{2} \right) h_w \qquad (2a)$$

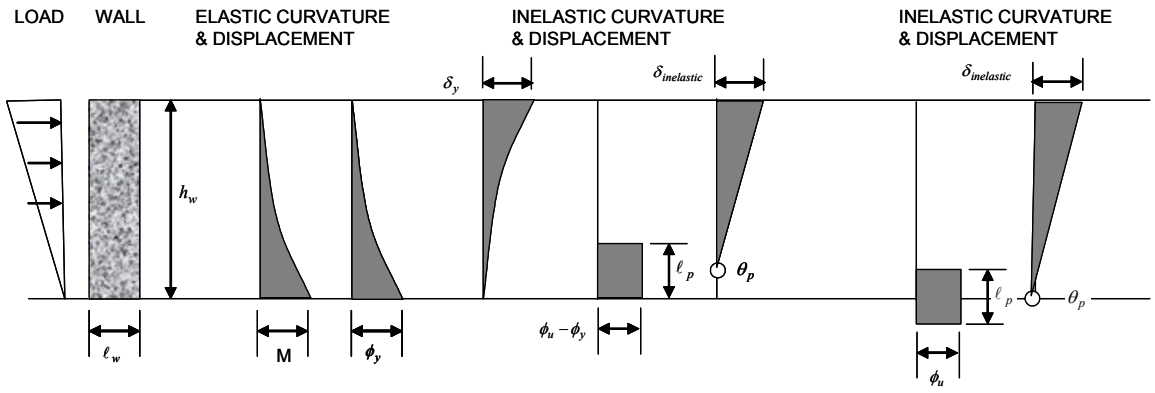
or, $$c_{cr} = \frac{\varepsilon_{mu}}{2} \frac{\ell_w}{(C_d \delta_{ne} / h_w)} \qquad (2b)$$

It follows from the above (see ~~Fig.~~Figure 3.3-4) that special detailing would be required if:

$$c \geq \frac{\varepsilon_{mu}}{2}\frac{\ell_w}{(C_d\delta_{ne}/h_w)} = \frac{0.003}{2}\frac{\ell_w}{(C_d\delta_{ne}/h_w)} = \frac{\ell_w}{667\,(C_d\delta_{ne}/h_w)} \approx \frac{\ell_w}{600\,(C_d\delta_{ne}/h_w)}$$

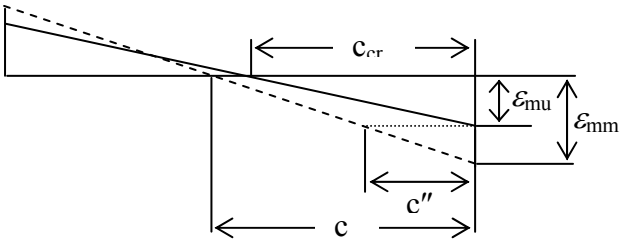
because if the neutral axis depth exceeded the critical value, the extreme fiber compressive strain would exceed the maximum usable strain ε_{mu}. For purposes of this derivation, and to avoid having separate sets of drift-related requirements for clay and concrete masonry, a single useful strain of 0.003 is used, representing an average of the design values of 0.0025 for concrete masonry and 0.0035 for clay masonry. In ACI 318-99, the term $(C_d\delta_{ne}/h_w)$ must equal or exceed 0.007. According to Wallace[3.20~~11~~12], "This lower limit on the mean drift ratio is included to ensure that ~~all~~ walls controlled by flexure have modest deformation capacities, as well as to guard against modeling errors that might underestimate the design displacement." This lower limit on $(C_d\delta_{ne}/h_w)$ has not been adopted for reinforced masonry walls because:

- 0.007 is arbitrary and appears to be too high for a system with a maximum drift of 0.01;
- 1997 UBC concrete provisions do not include this requirement; and
- many designs are already stiff, since masonry has never had boundary elements. Furthermore, stiffening the structure is a reasonable design alternative that should not be precluded (or limited). Further background related to concrete masonry shear walls is provided in References 3.~~12~~2~~11~~13, 3.22~~13~~14, and 3.23~~14~~15.



(a) Theoretical model (b) Simplified Model

~~Fig.~~Figure 3.3-3– Wall curvature and displacement



~~Fig.~~ Figure 3.3-4 —Strain distribution at critical section

(b) Where special detailing is required at the wall boundary, it must be extended vertically a distance not less than the larger of l_w and $M_u / 4V_u$ from the critical section. These lengths, also specified in ACI 318-99, where intended to be an upper-bound estimate of the plastic hinge length for special reinforced concrete shear walls. The same lengths have been adopted for intermediate and special masonry shear walls.

3.3.6.9 A stress-based approach was included in ACI 318-99 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). Maintaining the stress-based approach also provided continuity between ACI 318-99 and earlier editions of ACI 318; however, modifications were introduced to address major shortcomings of the design approach in pre-1999 editions of ACI 318.

The stress limit at which special detailing is required at the boundaries of reinforced concrete shear walls was left unchanged in ACI 318-99 at $0.2\ f'_c$, a value carried over from prior editions of the Code. The special detailing, where required, must be extended over the height of the wall from the critical section until the calculated stress drops below $0.15\ f'_c$, once again the same value as in prior editions of ACI 318.

A major difference between ACI 318-99 and prior editions of ACI 318 is in the way a shear wall requiring specially detailed boundary elements is to be designed for flexure and axial loads. ACI 318-95 required that the boundary elements be designed to resist (as short columns) the tributary gravity load plus the compressive resultant associated with the overturning moment at the base of the wall (both taken at factored values). The application of this requirement typically resulted in safe boundary elements containing high percentages of reinforcement, resulting in a substantial increase in wall flexural strength. Constructability suffered as a result, but more

importantly, brittle shear failure preceding ductile flexural failure became more likely, because walls having excessive flexural strength would draw larger shear forces in an earthquake event, and the Code did not require shear strength to be increased proportionally with the increase in flexural strength. ACI 318-99 does not require the boundary elements to resist the entire P_u and M_u even when the stress-based approach is used. In fact, a shear wall is designed in exactly the same way for flexure and axial load, irrespective of whether the displacement-based approach or the stress-based approach is used to trigger special boundary elements.

The 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 $0.2f'_c$ to $0.15f'_c$ for reinforced concrete walls to $0.2f'_m$ to $0.15f'_m$, 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.

3.3.6.10 Unlike in the case of concrete, where prescriptive detailing requirements for the specially confined boundary element are given in ACI 318-99, this proposal requires that testing be done to verify that the detailing provided shall be capable of developing a strain capacity in the boundary element that would be in excess of the maximum imposed strain. It is hoped that reasonably extensive tests will be conducted in the near future, leading to the development of prescriptive detailing requirements for specially confined boundary elements of intermediate as well as special reinforced masonry shear walls.

(a) Figure 3.3-4 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 ~~Fig.~~Figure 3.3-3(b),

$$C_d\delta_{ne} = \theta_p h_w = (\phi_u \ell_p)h_w = \left(\frac{\varepsilon_{mm}}{c}\frac{\ell_w}{2}\right)h_w \qquad (3)$$

From Eq. (3):

$$\varepsilon_{mm} = 2\left(\frac{C_d \delta_{ne}}{h_w}\right)\left(\frac{c}{\ell_w}\right) \qquad (4)$$

The wall length over which the strains exceed the limiting value of ε_{mu}, denoted as c'', can be determined using similar triangles from ~~Fig.~~Figure 3.3-4:

$$c'' = c\left(1 - \frac{\varepsilon_{mu}}{\varepsilon_{mm}}\right) \qquad (5)$$

An expression for the required length of confinement can be developed by combining Eqs. (2) and (3):

$$\frac{c''}{\ell_w} = \frac{c}{\ell_w} - \frac{(\varepsilon_{mu}/2)}{(C_d \delta_{ne}/h_w)} \qquad (6)$$

The term c/ℓ_w in Eq. (4) accounts for the influence of material properties (f'_m, f_y), axial load, geometry, and quantities and distribution of reinforcement, whereas the term $(\varepsilon_{mu}/2)/(C_d \delta_{ne}/h_w)$ accounts for the influence of system response (roof displacement) and the maximum usable strain of masonry.

The wall length over which special transverse reinforcement must be provided is based on Eq. (6), with a value of $C_d \delta_{ne}/h_w = 0.015$:

$$\frac{c''}{\ell_w} = \frac{c}{\ell_w} - \frac{(0.003/2)}{0.015} = \frac{c}{\ell_w} - 0.1 \geq \frac{c}{2} \qquad (7)$$

The value of $C_d \delta_{ne}/h_w$ was selected to provide an upper-bound estimate of the mean drift ratio of typical shear wall buildings constructed in the United States of America[3.~~14~~16~~23~~]. Thus, the length of the wall that must be confined ~~tends toward the~~is conservative ~~side~~ for many buildings. The value of $c/2$ represents a minimum length of confinement, is adopted from ACI 318-99, and is arbitrary.

(b) This requirement originated in the 1997 UBC and has been carried over into ACI 318-99 and -02. Where flanges are heavily stressed in compression, the web-to-flange interface is likely to be heavily stressed and may sustain local crushing failure unless special boundary element reinforcement extends into the web.

(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) 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. According to the Commentary to ACI 318, achievement of this anchorage is difficult when large transverse cracks occur in the boundary elements. That Commentary recommends the use of standard 90-degree hooks or mechanical anchorage schemes, instead of straight bar development.

References

3.1 Brown, R.H. and Whitlock, A.R., "Strength of Anchor Bolts in Concrete Masonry," Journal of the Structural Division, American Society of Civil Engineers, New York, NY, Vol. 109, No. 6, June, 1983, pp. 1362-1374.

3.2 Hatzinikolos, M., Longworth, J., and Warwaruk, J., "Strength and Behavior of Anchor Bolts Embedded in Concrete Masonry," Proceedings, 2nd Canadian Masonry Conference, Carleton University, Ottawa, Ontario, June, 1980. pp. 549-563.

3.3 Rad, F. N., Muller, W. H. and Winnen, J. M., "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.

3.4 Tubbs, J. B., Pollock, D. G., Jr., McLean, D. I. and Young, T. C. (1999), "Performance of Anchor Bolts in Concrete Block Masonry", Proceedings, 8th North American Masonry Conference, Austin, Texas, June 6-9, 1999.

3.5 Allen, R., Borchelt, J. G., Klingner, R. E. and Zobel, R., "Proposed Provisions for Design of Anchorage to Masonry," The Masonry Society Journal, Vol. 18, No. 2, Dec. 2000, pp. 35-59.

3.6 Brown, R. H., Borchelt, J. G., and Burgess, R. E., "Strength of Anchor Bolts in the Top of Clay Masonry Walls," Proceedings of the 9th Canadian Masonry Symposium, Fredericton, New Brunswick, Canada, June 2001.

3.7 Weigel, T. A., Mohsen, J. P., Burke, A., Erdmann, K. and Schad, A., "Tensile Strength of Headed Anchor Bolts in Tops of CMU Walls," The Masonry Society Journal, Vol. 20, No. 1, December 2002, pp 61-70.

3.8 ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)", American Concrete Institute, Farmington Hills, MI.

3.9~~1~~. Malik, J. B., Mendonca, J. A., and Klingner, R. E., "Effect of Reinforcing Details on the Shear Resistance of Short Anchor Bolts under Reversed Cyclic Loading," Journal of the American Concrete Institute, Proceedings Vol. 79, No. 1, January-February 1982, pp. 3-11.

3.10~~2~~. The following Technical Coordinating Committee for Masonry Research (TCCMaR) task reports not specifically cited in this Chapter provide the substantiating data for the strength design criteria presented.

Report No. 1.1-1: Atkinson and Kingsley, *Comparison of the Behavior of Clay & Concrete Masonry in Compression*, September 1985, 151 pgs.

Report No. 1.2(a)-l: Hamid, A. A., Assis, G.F., and Harris, H.G., *Material Models for Grouted Block Masonry*, August 1988, 67 pgs.

Report No. 1.2(a)-2: Assis, G. F., Hamid, A.A., and Harris, H.G., *Material Models for Grouted Block Masonry*, August 1989, 134 pgs.

Report No. 1.2(b)-l: Young, J. M., and Brown, R.H., *Compressive Stress Distribution of Grouted Hollow Clay Masonry Under Strain Gradient*, May 1988, 170 pgs.

Report No. 1.3-1: Atkinson, R. H., *An Assessment of Current Material Test Standards for Masonry Limit States Design Methods*, June 1991, 38 pgs.

Report No. 2.1-1: Hart, G., and Basharkhah, M., *Slender Wall Structural Engineering Analysis Computer Program* (*Shwall*, Version 1. 01), September 1987. 68 pgs.

Report No. 2.1-2: Hart, G., and Basharkhah, M., *Shear Wall Structural Engineering Analysis Computer Program* (*Shwall*, Version 1.01), September 1987, 75 pgs.

Report No. 2.1-3: Nakaki, D., and G. Hart, *Uplifting Response of Structures Subjected to Earthquake Motions*, August 1987, 200 pgs.

Report No. 2.1-4: Hart, G., Sajjad, N., and Basharkhah, M., *Inelastic Column Analysis Computer Program* (*INCAP*, Version 1.01), March 1988.

Report No. 2.1-5: Hong, W. K., Hart, G.C., and Englekirk, R.E., *Force-Deflection Evaluation and Models for University of Colorado Flexural Walls*, December 1989.

Report No. 2.1-6: Hart, G. C., Jaw, J.W., and Low, Y.K., SC*M Model for University of Colorado Flexural Walls*, December 1989, 31 pgs.

Report No. 2.1-7: Hart, G. C., Sajjad, N., and Basharkhah, M., *Inelastic Masonry Flexural Shear Wall Analysis Computer Program*, February 1990, 41 pgs.

Report No. 2.1-8: Hart, G. C., Englekirk, R.E., Srinivasan, M., Huang, S.C., and Drag, D.J., *Seismic Performance Study, DPC Gymnasium, Elastic Time History Analysis Using SAP90*, February 1992, 41 pgs.

Report No. 2.1-9: Hart, G. C., Englekirk, R.E., Srinivasan, M., Huang, S.C., and Drag, D.J., *Seismic Performance Study, TMS Shopping Center, Elastic Time History Analysis Using SAP90*, February 1992, 42 pgs.

Report No. 2.1-10: Hart, G. C., Englekirk, R.E., Jaw, J.W., Srinivasan, M., Huang, S.C., and Drag, D.J., *Seismic Performance Study, RCJ Hotel*, February 1992, 51 pgs.

Report No. 2.1-11: Hart, G. C., Englekirk, R.E., Srinivasan, M., Huang, S.C., and Drag, D.J., *Performance Study, 2-Story Masonry Wall-Frame Building*, February 1992, 112 pgs.

Report No. 2.1-12: Hart, G. C., Englekirk, R.E., Jaw, J.W., Srinivasan, M., Huang, S.C., and Drag, D.J., *Seismic Performance Study, Designed by Tentative Limit Sates Design Standard*, February 1992, 75 pgs.

Report No. 2.2-1: Ewing, R. D., A. El-Mustapha, and Kariotis, J., *FEM/I - A Finite Element Computer Program for the Nonlinear Static Analysis of Reinforced Masonry Building Components*, December 1987 (Revised June 1990), 124 pgs.

Report No. 2.2-2: Ewing, R. D., *Parametric Studies on Reinforced Masonry Shear Walls Using FEM/I, A Nonlinear Finite Element Analysis Program*, March 1992.

Report No. 2.2-3: Ewing, R. D., *Finite Element Analysis of Reinforced Masonry Building Components Designed by a Tentative Masonry Limit States Design Standard*, March 1992, 48 pgs.

Report No. 2.3-1: Ewing, R., J. Kariotis, and A. El-Mustapha, *LPM/I, A Computer Program for the Nonlinear, Dynamic Analysis of Lumped Parameter Models*, August 1987, 200 pgs.

Report No. 2.3-2: Kariotis, J., El-Mustapha, A., and Ewing, R., *Influence of Foundation Model on the Uplifting of Structures*, July 1988, 50 pgs.

Report No. 2.3-3: Kariotis, J., Rahman, A., and El-Mustapha, A., *Investigation of Current Seismic Design Provisions for Reinforced Masonry Shear Walls*, January 1990, 48 pgs.

Report No. 2.3-4: Kariotis, J., Rahman, A., Waqfi, O., and Ewing, R., *Version 1.03 LPM/I - A Computer Program for the Nonlinear, Dynamic Analysis of Lumped Parameter Models*, February 1992, 227 pgs.

Report No. 2.3-5: Kariotis, J., Waqfi, O., and Ewing, R., *A Computer Program Using Beam Elements for the Nonlinear, Dynamic Analysis of Lumped Parameter Models*, February 1992, 96 pgs.

Report No. 2.3-6: Kariotis, J., and Waqfi, O., *Comparison of the Dynamic Response of a Damped MDOF Nonlinear Beam Model with an Equivalent SDOF Hysteretic Model*, April 1992, 88 pgs.

Report No. 2.3-7: Kariotis, J., and Waqfi, O., *Recommended Procedure for Calculation of the Balanced Reinforcement Ratio*, February 1992, 73 pgs.

Report No. 2.4(b)-1: Button, M. R., and Mayes, R.L., *Out-of-Plane Seismic Response of Reinforced Masonry Walls: Correlation of Full-Scale Test and Analytical Model Results*, March 1991, 65 pgs.

Report No. 3.1(a)-1: Scrivener, J., *Summary of Findings of Cyclic Tests on Masonry Piers*, June 1986, 7 pgs.

Report No. 3.1(a)-2: Shing, P. B., Noland, J., Spaeh, H., Klamerus, E., and Schuller, M., *Response of Single-Story Reinforced Masonry Shear Walls to In-Plane Lateral Loads*, January 1991, 136 pgs.

Report No. 3.1(b)-1: Seible, F., and LaRovere, H., *Summary of Pseudo Dynamic Testing*, February 1987, 46 pgs.

Report No. 3.1(b)-2: Igarashi, A., Seible, F., and Hegemier, G., *Development of the Generated Sequential displacement Procedure and the Simulated Seismic Testing of the TCCMaR Three-Story In-Plane Walls*, June 1993.

Report No. 3.1(c)-1: Merryman, K., Leiva, G., Antrobus, B., and Klingner, R., *In-Plane Seismic Resistance of Two-Story Concrete Masonry Coupled Shear Walls*, September 1989, 176 pgs.

Report No. 3.1(c)-2: Leiva, G., and Klingner, R., *In-plane Seismic Resistance of Two-story Concrete Masonry Shear Walls with Openings*, August 1991, 326 pgs.

Report No. 3.2(a)-1: Hamid, A., Abboud, B., Farah, M., Hatem, K., and Harris, H., *Response of Reinforced Block Masonry Walls to Out-of-Plane Static Loads*, September 1989, 120 pgs.

Report No. 3.2(b)-1: Agbabian, M., Adham, S., Masri, S., Avanessian, V., and Traina, V., *Out-of-Plane Dynamic Testing of Concrete Masonry Walls*, Vol. 1 and 2, July 1989, 220 pgs.

Report No. 3.2(b)-2: Blondet, M., and Mayes, R.L., *The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes, Vol. 1: General Information*, April 1991, 172 pgs.

Report No. 3.2(b)-2: Blondet, M., and Mayes, R.L., *The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes, Vol. 2: Walls No. 4 and 6 (Group 1)*, April 1991, 267 pgs.

Report No. 3.2(b)-2: Blondet, M., and Mayes, R.L., *The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes, Vol. 3: Walls No. 8, 9, 10 and 11 (Group 2)*, April 1991, 310 pgs.

Report No. 3.2(b)-2: Blondet, M., and Mayes, R.L., *The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes, Vol. 4: Walls No. 3, 5, and 7 (Group 3)*, April 1991, 256 pgs.

Report No. 4.1-1: He, L., and Priestley, M.J.N., *Seismic Behavior of Flanged Masonry Shear Walls*, May 1988, 119 pgs.

Report No. 4.1-2: He, L., and Priestley, M.J.N., *Seismic Behavior of Flanged Masonry Shear Walls -Final Report*, November 1992, 279 pgs.

Report No. 4.2-1: Hegemier, G., and Murakami, H., *On the Behavior of Floor-to-Wall Intersections in Concrete Masonry Construction: Part I: Experimental.*

Report No. 4.2-2: Hegemier, G., and Murakami, H., *On the Behavior of Floor-to-Wall Intersections in Concrete Masonry Construction: Part II: Theoretical.*

Report No. 5.1-1: Porter, M., and Sabri, A., *Plank Diaphragm Characteristics*, July 1990, 226 pgs.

Report No. 5.2-1: Porter, M., Yeomans, F., and Johns, A., *Assembly of Existing Diaphragm Data*, July 1990, 142 pgs.

Report No. 6.2-1: Scrivener, J., *Bond of Reinforcement in Grouted Hollow Unit Masonry: A State-of-the-Art*, June 1986, 53 pgs.

Report No. 6.2-2: Soric, Z., and Tulin, L., *Bond Splices in Reinforced Masonry*, August 1987, 296 pgs.

Report No. 7.1-1: Paulson, T., and Abrams, D., *Measured Inelastic Response of Reinforced Masonry Building Structures to Earthquake Motions*, October 1990, 294 pgs.

Report No. 8.1-1: Hart, G., *A Limit State Design Method for Reinforced Masonry*, June 1988.

Report No. 8.1-2: Hart, G., *Expected Value Design in the Context of a Limit Sate Design Methodology*, February 1990.

Report No. 8.2-1: Hart, G., and Zorapapel, G.T., *Reliability of Concrete Masonry Wall Structures*, December 1991, 229 pgs.

Report No. 8.2-2: Hart, G., and Sajjad, N., *Confinement in Concrete Masonry*, December 1990.

Report No. 8.2-3: Hart, G., and Jang, J., *Seismic Performance of Masonry Wall Frames*, December 1991.

Report No. 9.1-1: Kariotis, J. C., and Johnson, A.W., *Design of Reinforced Masonry Research Building*, September 1987, 42 pgs.

Report No. 9.1-2: Kariotis, J. C., and Waqfi, O.M., *Trial Designs Made in Accordance with Tentative Limit States Design Standards for Reinforced Masonry Buildings*, February 1992, 184 pgs.

Report No. 9.2-1: Seible, F., *Report on Large Structures Testing Facilities in Japan*, September 1985, 120 pgs.

Report No. 9.2-2: Seible, F., *Design and Construction of the Charles Lee Powell Structural Systems Laboratory*, November 1986, 65 pgs.

Report No. 9.2-3: Seible, F., *The Japanese Five-story Full Scale Reinforced Masonry Building Test*, January 1988, 100 pgs.

Report No. 9.2-4: Seible, F., Hegemier, G.A., Priestley, M.J.N., Kingsley, G.R., Kurkchubasche, A., and Igarashi, A. *The U.S. - TCCMAR Five-story Full Scale Masonry Research Building Test - Preliminary Report*, October 1992, 58 pgs.

Report No. 11.1-1: *TCCMaR, Summary Report: U. S. Coordinated Program for Masonry Building Research*, September 1985 to August 1986, 190 pgs.

Report No. 11.1-2: *TCCMaR, Status Report - U.S. Coordinated Program for Masonry Building Research*, November 1988, 170 pgs.

.

3.113 Mirza, S.A., Lee, P.M., and Morgan, D.L. (1987). "ACI stability resistance factor for RC columns." Journal of Structural Engineering, ASCE, 113(9), 1963-1976.

3.124 MacGregor, J.G., Breen, J.E., and Pfrang, E.O. (1970). "Design of slender concrete columns." ACI Journal, 67(1), 6-28.

3.~~3~~135~~5~~ Assis, G.F. and Hamid, A.A., *Compression Behavior of Concrete Masonry Prisms Under Strain Gradient*, The Masonry Society Journal, 1990.

3.1464~~4~~. Brown, R.H., *Compressive Stress Distribution of Grouted Hollow Clay Masonry Under Strain Gradient*, The Masonry Society Journal, 1987.

3.~~5~~157~~7~~ National Concrete Masonry Association, "Evaluation of Reinforcing Bar Splice Criteria for Hollow Clay Brick and Hollow Concrete Block Masonry," Herndon, VA, May, 1999.

3.~~6~~168~~8~~. Thompson, J.J., "Behavior and Design of Tension Lap Splices in Reinforced Concrete Masonry," Masters Thesis, Department of Civil and Environmental Engineering, Washington State University, Pullman, Washington, 1997.

3.7179. Hammons, M.I., Atkinson, R.H., Schuller, M.P., Tikalsky, P.J., "Masonry Research for Limit-States Design," Construction Productivity Advancement Research Program Technical Report, CPAR-SL-94-1, October 1994, 136pp.

3.8180. Borchelt, J.G. and J.L. Elder, "Reinforcing Bar Splices in Hollow Brick Masonry," Proceedings of the 11th International Brick/Block Masonry Conference, Tongji University, Shanghai, China, October 1997, pp. 306-316.

3.9191. Hogan, M.B., Samblanet, P.J., and Thomas, R.D., "Research Evaluation of Reinforcing Bar Splices in Concrete Masonry," Proceedings of the 11th International Brick/Block Masonry Conference, Tongji University, Shanghai, China, October 1997, pp. 227-238

~~3.10. Branson, Dan E., "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, 1965, pp.1-78.~~

3.11~~20~~12. Wallace, J.W. and Orakcal, K., "ACI 318-99 Provisions for Seismic Design of Structural Walls," ACI Structural Journal, Vol. 99, No. 4, July-August 2002.

3.12~~21~~13. Paulay, T., "The Design of Ductile Reinforced Concrete Structural Walls for Earthquake Resistance," Earthquake Spectra, EERI, Vol. 2, No. 4, 1986, pp. 783-823.

3.13~~22~~15. Wallace, J.W., "A New Methodology for Seismic Design of RC Shear Walls," Journal of Structural Engineering, ASCE, Vol. 120, No. 3, 1994, pp. 863-884.

3.14~~23~~16. Wallace, J.W. and Moehle, J.P., "Ductility and Detailing Requirements of Bearing Wall Buildings," Journal of Structural Engineering, ASCE, Vol. 118, No. 6, 1992, pp. 1625-1644.

CHAPTER 4
PRESTRESSED MASONRY

4.1 — General

4.1.1 *Scope*

Prestressing forces are used in masonry walls 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). Since most research and applications to date have focused on walls, the chapter applies only to walls, not columns, beams, nor lintels. (Provisions for columns, beams, and lintels will be developed in future editions of the Code.)

Most construction applications to date have involved post-tensioned, ungrouted 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 elsewhere[4.1].

Throughout this Code and Specification, references to "reinforcement" apply to non-prestressed reinforcement. These references do not apply to prestressing tendons, except as explicitly noted in Chapter 4. Requirements for prestressing tendons use the terms "prestressing tendon" or "tendon." The provisions of Chapter 4 do not require a mandatory quantity of reinforcement or bonded prestressing tendons for prestressed masonry walls.

Anchorage forces are distributed within a wall similar to the way in which concentrated loads are distributed (as described in ~~Chapter 2~~Section 1.~~1~~9.7; see Figure ~~2.1-14~~1.~~1~~9-7~~1~~). However, research[4.24] has indicated that prestress losses can distribute to adjacent tendons as far laterally from the anchorage as the height of the wall.

4.2 — Design methods

Originally, prestressed masonry was designed using allowable stress design with a moment strength check for walls with laterally restrained tendons. The British code for prestressed

masonry[4.2, 4.3] 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[4.4 - 4.8]. Design methods are now based upon strength provisions with serviceability checks.

Often, a masonry wall is prestressed prior to 28 days 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.

4.3 — Permissible stresses in prestressing tendons

Allowable, prestressing-tendon stresses are based on criteria established for prestressed concrete[4.9]. 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 computing 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.

4.3.4 *Effective prestress*

The state of stress in a prestressed masonry wall must be checked for ~~all~~ each stage~~s~~ of loading. For each loading condition, the effective level of prestress should be used in the computation of stresses and wall 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[4.10 - 4.13].

Instantaneous deformation of masonry due to the application of prestress may be computed by the modulus of elasticity of masonry given in Section 1.8.2. Creep, shrinkage, and moisture expansion of masonry may be computed by the coefficients given in Section 1.8. 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 stressed to a large fraction of its yield stress 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. ASTM A 416, A 421, and A 722[4.14, 4.15, 4.16]prestressing steels 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 phenomenon such as creep, shrinkage, and moisture expansion of the masonry. An appropriate formula for predicting prestress loss due to relaxation has been developed[4.11 — 4.13]. Alternately, direct addition of the steel stress-relaxation value provided by the manufacturer can be used to compute 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 compute 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 has indicated that prestress losses are expected to be in the following ranges[4.22, 4.24-4.26]:

(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, unless calculations, experience, or construction techniques indicate different losses are expected.

4.4 — Axial compression and flexure

4.4.1 *General*

The requirements for prestressed masonry walls 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 prestressed masonry wall significantly depends 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. Alternatively, lateral restraint may be provided by building the masonry into contact with the tendon or the protective 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 wall 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 and reinforced masonry in Sections 2.2 and 2.3, respectively. 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 wall 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 2.2. 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.

4.4.2 *Service load requirements*

Since masonry walls with laterally unrestrained prestressing tendons are equivalent to masonry walls subjected to applied axial loads, the design approach for unreinforced masonry in Section 2.2 has been adopted for convenience and consistency. Buckling of masonry walls under prestressing force must be avoided for walls with laterally unrestrained prestressing tendons. The prestressing force, P_{ps}, is to be added to the design axial load, P, for ~~all~~ stress and load computations and in the computation of the eccentricity of the axial resultant, e.

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. Alternatively, lateral restraint may be provided by building the masonry into contact with the tendon or the tendon's protective sheath at periodic intervals along the length of the prestressing tendon. In general, three intermediate contacts within a laterally unsupported wall length or height can be considered to provide full lateral support of the tendon.

Prestressed masonry walls with laterally restrained prestressing tendons require a modified design approach from the criteria in Section 2.2. If the prestressing tendon is laterally restrained, the wall 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 walls are susceptible to buckling under axial loads other than prestressing, however, and this loading condition must be checked.[4.17] For this condition, with both concentrically and eccentrically prestressed masonry walls, the prestressing force must be considered in the computation of the eccentricity of this axial resultant, e, in Eq. (2-~~15~~18) of the Code. The flexural stress induced by eccentric prestressing causes an increase or decrease in the axial buckling load, depending upon the location and magnitude of the applied axial load relative to the prestressing force.

4.4.3 *Strength requirements*

Computation of the moment strength of prestressed masonry walls is similar to the method for prestressed concrete.[4.9] For bonded tendons, the simplification of taking the tendon stress at nominal moment strength equal to the yield stress can be more conservative for bars than for strands because

the yield stress of a prestressing bar is a smaller percentage of the ultimate strength of the tendon.

The response modification coefficient (R) and deflection amplification factor (C_d) used for unreinforced masonry are also used in the design of prestressed masonry. This requirement ensures that the structural response of prestressed masonry structures, designed in accordance with these provisions, will essentially remain in the elastic range. When more experimental and field data are available on the ductility of both unbonded and bonded systems, R and C_d factors can be reviewed.

The equations for the unbonded prestressing tendon stress, f_{ps}, at the moment strength condition (Eq. 4-3 and 4-4) are based on tests of prestressed masonry walls, which were primarily loaded out-of-plane. Equation (4-3) is used for calculating tendon stress at nominal moment capacity for members ~~with~~ loaded out-of-plane and which contain laterally restrained tendons. Equation (4-4) is provided for calculating stresses at moment strength for unbonded, unrestrained tendons, when the wall is loaded out-of-plane. These equations provide improved estimates of the tendon stresses at ultimate capacity over previous equations in the Code[4.20, 4.21].

The equation for the nominal moment strength, M_n, is for the general case of a masonry 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.

The depth of the equivalent compression ~~zone~~ stress block must be determined with consideration of the cross section of the wall, the tensile resistance of tendons and reinforcement, and the factored design axial load, P_u. P_u is an additive quantity in Code Eqs. (4-1) and (4-2). Prestressing adds to the resistance for ultimate strength evaluations and is used with a load factor of 1.0. Equation (4-1) defining the depth of the equivalent compression ~~zone~~stress block, a, is modified to match the value for the equivalent uniform stress parameter specified in Chapter 3 (Strength Design of Masonry) of the Code ($0.80 f'_m$). A review of existing tests of post-tensioned masonry walls indicates that the flexural strength of the walls is more accurately calculated using uniform stresses smaller than the value specified in Chapter 4 in previous editions of the Code ($0.85 f'_m$)[4.20, 4.21].

The ratio, *a/d*, must be less than 0.425 to promote ductile performance in flexure. This 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 0.8.

4.5 — Axial tension

The axial tensile strength of masonry in a prestressed masonry wall is to be neglected, which is a conservative measure. This requirement is consistent with that of Section 2.3. If axial tension develops, for example due to wind uplift on the roof structure, the axial tension must be resisted by reinforcement, tendons, or both.

4.6 — Shear

This section applies to both in-plane and out-of-plane shear.

The shear capacity of prestressed walls is calculated using the provisions of the Chapter 3. Calculation of shear capacity is dictated by the presence or absence of bonded mild reinforcement. While the MSJC acknowledges that prestressed masonry walls are reinforced, for walls without bonded mild reinforcement, the unreinforced (plain) masonry shear provisions of Chapter 3 are used to calculate shear capacity. When bonded mild reinforcement is provided, then the reinforced masonry shear provisions of Chapter 3 are used to calculate shear capacity.

No shear strength enhancement due to arching action of the masonry is recognized in this Code for prestressed masonry walls. The formation of compression struts and tension ties in prestressed masonry is possible, but this phenomenon has not been considered.

4.7 — Deflection

In accordance with Chapter 1, prestressed masonry wall deflection should be computed based on uncracked section properties. Computation of wall deflection must include the effect of time-dependent phenomenon such as creep and shrinkage of masonry and relaxation of prestressing tendons. There are no limits for the out-of-plane deflection of prestressed masonry walls. This is because appropriate out-of-plane deflection limits are project-specific. The designer should consider the potential for damage to interior finishes, and should limit deflections accordingly.

4.8 — 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

masonry wall without distress to the masonry or the prestressing accessories. Anchorages are designed for adequate pull-out strength from their foundations.

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[4.18] 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 the references[4.19].

4.9 — Protection of prestressing tendons and accessories

Corrosion protection of the prestressing tendon and accessories is required in masonry walls subject to a moist and corrosive environment. Methods of corrosion protection are addressed in the Specification. 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 the Specification provide a minimum level of corrosion protection. The designer may wish to impose more substantial corrosion protection requirements, especially in highly corrosive environments.

4.10 — 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.

References

4.1. Schultz, A.E. and Scolforo, M.J., "An Overview of Prestressed Masonry," *The Masonry Society Journal*, V. 10, No. 1, The Masonry Society, Boulder, CO, August 1991, pp. 6-21.

4.2. *Code of Practice for the Use of Masonry, Part 2: Reinforced and Prestressed Masonry*, BS 5628, British Standards Institution, London, England, 1985.

4.3. Phipps, M.E., "The Codification of Prestressed Masonry Design," *Proceedings*, Sixth Canadian Masonry Symposium, Saskatoon, Saskatchewan, Canada, June 1992, pp. 561-572.

4.4. Schultz, A.E. and Scolforo, M.J., "Engineering Design Provisions for Prestressed Masonry, Part 1: Masonry Stresses," *The Masonry Society Journal*, V. 10, No. 2, The Masonry Society, Boulder, CO, February 1992, pp. 29-47.

4.5. Schultz, A.E., and Scolforo, M.J., "Engineering Design Provisions for Prestressed Masonry, Part 2: Steel Stresses and Other Considerations," *The Masonry Society Journal*, V. 10, No. 2, The Masonry Society, Boulder, CO, February 1992, pp. 48-64.

4.6. *Post-Tensioned Masonry Structures*, VSL International Ltd., VSL Report Series, Berne, Switzerland, 1990, 35 pp.

4.7. Curtin, W.G., Shaw, G., and Beck, J.K., *Design of Reinforced and Prestressed Masonry*, Thomas Telford Ltd., London, England, 1988, 244 pp.

4.8. Phipps, M.E. and Montague, T.I., "The Design of Prestressed Concrete Blockwork Diaphragm Walls," *Aggregate Concrete Block Association*, England, 1976, 18 pp.

4.9. *Building Code Requirements for Reinforced Concrete*, ACI 318-95, American Concrete Institute, Detroit, MI, 1995.

4.10. "Recommendations for Estimating Prestress Losses," Report of PCI Committee on Prestress Losses, *Journal of the Prestressed Concrete Institute*, V. 20, No. 4, Chicago, IL, July-August 1975, pp. 43-75.

4.11. Lenczner, D., "Creep and Stress Relaxation in Stack-Bonded Brick Masonry Prisms, A Pilot Study," Department of Civil Engineering, Clemson University, Clemson, SC, May 1985, 28 pp.

4.12. Lenczner, D., "Creep and Loss of Prestress in Stack Bonded Brick Masonry Prisms, Pilot Study - Stage II," Department of Civil Engineering, University of Illinois, Urbana-Champaign, IL, August 1987, 29 pp.

4.13. Shrive, N.G., "Effects of Time Dependent Movements in Composite and Post-Tensioned Masonry," *Masonry International*, V. 2, No. 1, British Masonry Society, London, England, Spring 1988, pp. 1-34.

4.14. ASTM A 416-96, *Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete*, American Society for Materials and Testing, West Conshohocken, PA.

4.15. ASTM A 421-91, Sta*ndard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete*, American Society for Materials and Testing, West Conshohocken, PA.

4.16. ASTM A 722-96, *Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete*, American Society for Materials and Testing, West Conshohocken, PA.

4.17. Scolforo, M.J. and Borchelt, J.G., "Design of Reinforced and Prestressed Slender Masonry Walls," *Proceedings*, Innovative Large Span Structures, The Canadian Society of Civil Engineers, Montreal, Canada, July 1992, pp. 709-720.

4.18. "Guide Specifications for Post-Tensioning Materials," *Post-Tensioning Manual*, 5th Edition, Post-Tensioning Institute, Phoenix, AZ, 1990, pp. 208-216.

4.19. Sanders, D.H., Breen, J.E., and Duncan, R.R. III, "Strength and Behavior of Closely Spaced Post-Tensioned Monostrand Anchorages," Post-Tensioning Institute, Phoenix, AZ, 1987, 49 pp.

4.20. Schultz, A. E., Bean, J. R., and Stolarski, H. K., “Resistance of Slender Post-Tensioned Masonry Walls with Unbonded Tendons to Transverse Loading”, *Proceedings*, 9th North American Masonry Conference, Clemson, South Carolina, June 2003.

4.21 Bean, J.R. and Schultz A.E., “Flexural Capacity of Post-Tensioned Masonry Walls:Code Review and Recommended Procedure”, PTI Journal, Vol. 1, No. 1, January 2003, pp.28-44.

4.22 Stierwalt, D. D. and Hamilton III, H. R., “Restraint Effectiveness in Unbonded Tendons for Post-tensioned Masonry,” *ACI Structural Journal*, Nov/Dec 2000, Vol. 97, No. 6, pp. 840-848.

4.23 Hamilton III, H. R. and Badger, C. C. R., “Creep Losses in Post-Tensioned Concrete Masonry,” *TMS Journal*, Vol. 18, No. 1, July 2000, pp. 19-30.

4.24 Woodham, D. B. and Hamilton III, H. R., “Monitoring Prestress Losses in Post-Tensioned Concrete Masonry,” *Proceedings*, 9th North American Masonry Conference, Clemson, South Carolina, June 2003.

4.25 Biggs, D.T. and Ganz, H.R., "The Codification of Prestressed Masonry in the United States", *Proceedings,* Fifth International Masonry Conference, London, UK, October 1998, pp. 363-366.

4.26 NCMA TEK-14-20A, "Post-tensioned Concrete Masonry Wall Design", National Concrete Masonry Association.

CHAPTER 5
EMPIRICAL DESIGN OF MASONRY

5.1 — General

Empirical rules and formulas for the design of masonry structures were developed by experience. These are part of the legacy of masonry's long use, predating engineering analysis. Design is based on the condition that gravity loads are reasonably centered on the bearing walls and foundation piers. Figure 5.1-1 illustrates the location of the resultant of gravity loads on foundation piers. The effect of any steel reinforcement, if used, is neglected. The masonry should be laid in running bond. Specific limitations on building height, seismic, wind, and horizontal loads exist. Buildings are of limited height. Members not participating in the lateral force-resisting system of a building may be empirically designed even though the lateral force-resisting system is designed under Chapter 2.

These procedures have been compiled through the years[5.1-5.5]. The most recent of these documents[5.5] is the basis for this chapter.

Empirical design is a procedure of sizing and proportioning masonry elements. It is not design analysis. This procedure is conservative for most masonry construction. Empirical design of masonry was developed for buildings of smaller scale, with more masonry interior walls and stiffer floor systems than built today. Thus, the limits imposed are valid.

Since empirically designed masonry is based on the gross compressive strength of the units, there is no need to specify the compressive strength of masonry.

5.3 — Lateral stability

Lateral stability requirements are a key provision of empirical design. Obviously, shear walls must be in two directions to provide stability. Bearing walls can serve as shear walls. The height of an element refers to the shortest unsupported height in the plane of the wall such as the shorter of a window jamb on one side and a door jamb on the other. See ~~Fig.~~Figure 5.3-1 for cumulative length of shear walls. See ~~Fig.~~Figure 5.3-2 for diaphragm panel length to width ratio determination.

5.4 — Compressive stress requirements

These are average compressive stresses based on gross area using ~~actual~~ specified dimensions. The following conditions should be used as guidelines when concentrated loads are placed on masonry:

- For concentrated loads acting on the full wall thickness, the allowable stresses under the load may be increased by 25 percent.
- For concentrated loads acting on concentrically placed bearing plates greater than one-half but less than full area, the allowable stress under the bearing plate may be increased by 50 percent.

The course immediately under the point of bearing should be a solid unit or filled solid with mortar or grout.

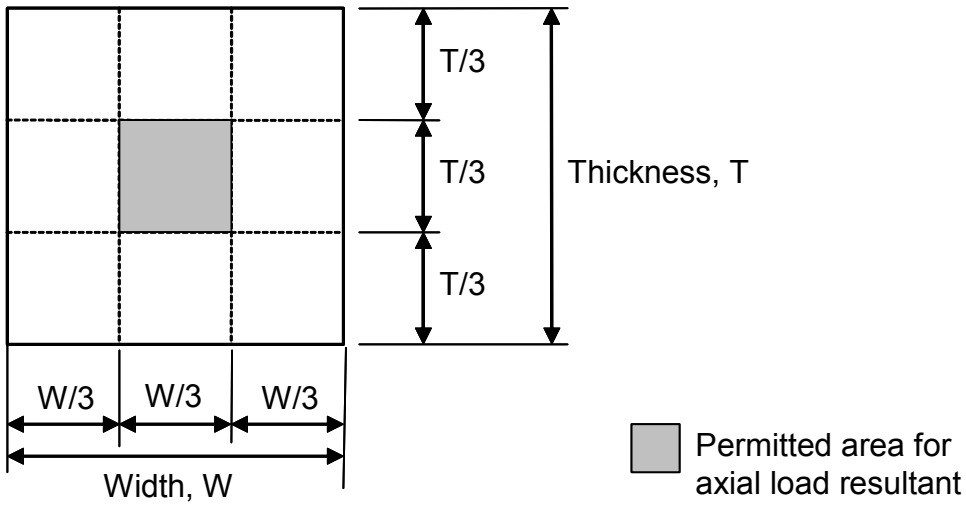


Figure 5.1-1 - Area for gravity loads applied to foundation piers

5.5 — Lateral support

Lateral support requirements are included to limit the flexural tensile stress due to out-of-plane loads. Masonry headers resist shear stress and permit the entire cross-section to perform as a single element. This is not the case for non-composite walls connected with wall ties. For such non-composite walls, the use of the sum of the thicknesses of the wythes has been used successfully for a long time and is a traditional approach that is acceptable within the limits imposed by Code Table 5.5.1. Requirements were added in the 2008 edition to provide relative out-of-plane resistance that limit the maximum width of opening and provide sufficient masonry sections between the openings.

5.6 — Thickness of masonry

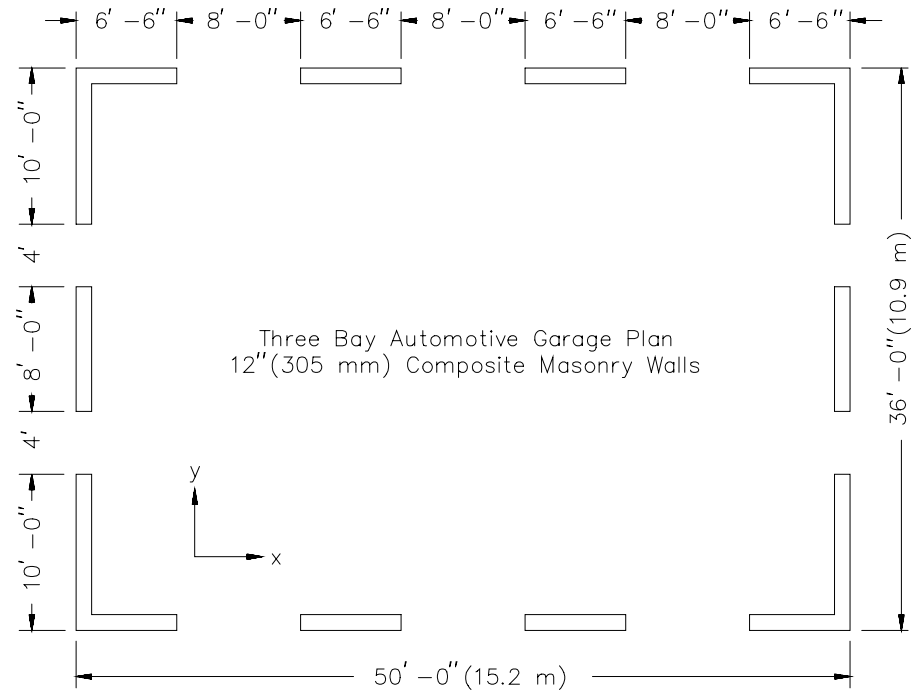
5.6.1 *General*

Experience of the committee has shown that the present ANSI A 41.1[5.5] thickness ratios are not always conservative. These requirements represent the consensus of the committee for more conservative design.

5.6.2.5 *Foundation piers* — Use of empirically designed foundation piers has been common practice in many areas of the country for many years. ANSI A 41.1[5.5] provisions for empirically designed piers (Section 5.3) includes a requirement for a maximum *h/t* ratio of 4. The minimum height-to-thickness ratio of greater than 4 for columns is required to clearly differentiate a column from a pier.

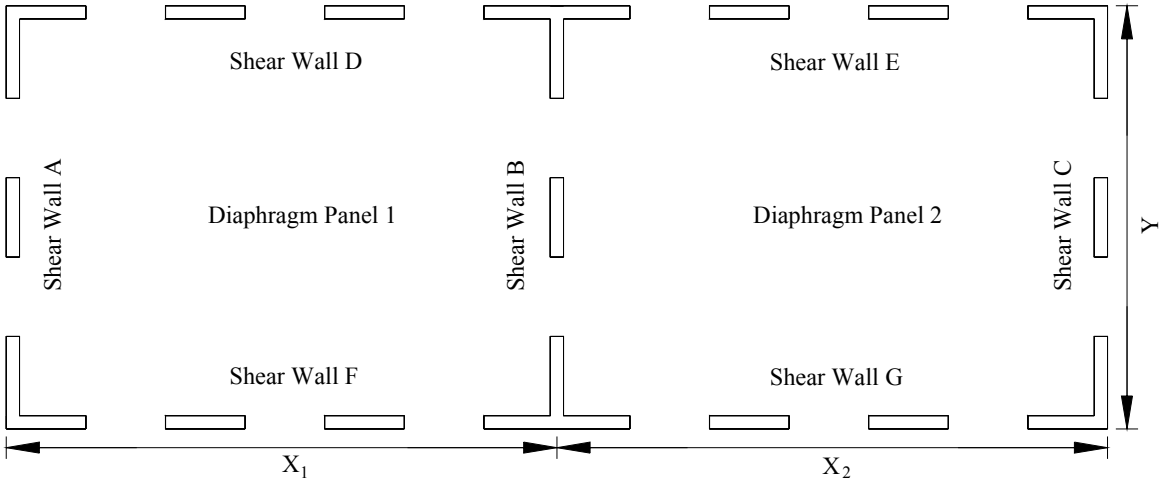
5.6.3 *Foundation walls*

Empirical criteria for masonry foundation wall thickness related to the depth of unbalanced fill have been contained in building codes and federal government standards for many years. The use of Code Table 5.6.3.1, which lists the traditional allowable backfill depths, is limited by a number of requirements that were not specified in previous codes and standards. These restrictions are enumerated in Section 5.6.3.1. Further precautions are recommended to guard against allowing heavy earth-moving or other equipment near enough to the foundation wall to develop high earth pressures. Experience with local conditions should be used to modify the values in Table 5.6.3.1 when appropriate.



x- Direction: ℓ = 2(6.5′ + 6.5′ + 6.5′ + 6.5′) = 52′ > 20′ OK
ℓ = 2(3.07 m + 3.07 m + 3.07 m + 3.07 m) = 15.9 m > 6.1 m OK
y- Direction: ℓ = 2(10′ + 8′ + 10′) = 56′ > 20′ OK
ℓ = 2(4.7 m + 3.38 m + 4.7 m) = 17.1 m > 6.1 m OK

~~Fig.~~Figure 5.3-1 — Cumulative length of shear walls



Diaphragm Panel Length = Dimension perpendicular to the resisting shear wall

Diaphragm Panel Width = Dimension parallel to the resisting shear wall

For example:

For Shear Walls A and B, the diaphragm panel length to width ratio is X_1/Y

For Shear Walls D and F, the diaphragm panel length to width ratio is Y/X_1

Note: Shear walls should be placed on all four sides of the diaphragm panel or the resulting torsion should be accounted for.

~~*Fig.*~~*Figure 5.3-2 — Diaphragm panel length to width ratio determination for shear wall spacing*

5.7 — Bond

~~Fig.~~Figure 5.7-1 depicts the requirements listed. Wall ties with drips are not permitted because of their reduced load capacity.

5.8 — Anchorage

The requirements of Sections 5.8.2.2 through 5.8.2.5 are less stringent than those of Section 1.9.4.2.5. Anchorage requirements in Section 5.8.3.3 are intended to comply with the Steel Joist Institute's Standard Specification[5.5] for end anchorage of steel joists.

References

5.1. Baker, I.O., *A Treatise on Masonry Construction*, University of Illinois, Champaign, IL, 1889, 1899, 1903. Also, 10th Edition, John Wiley & Sons, New York, NY, 1909, 745 pp.

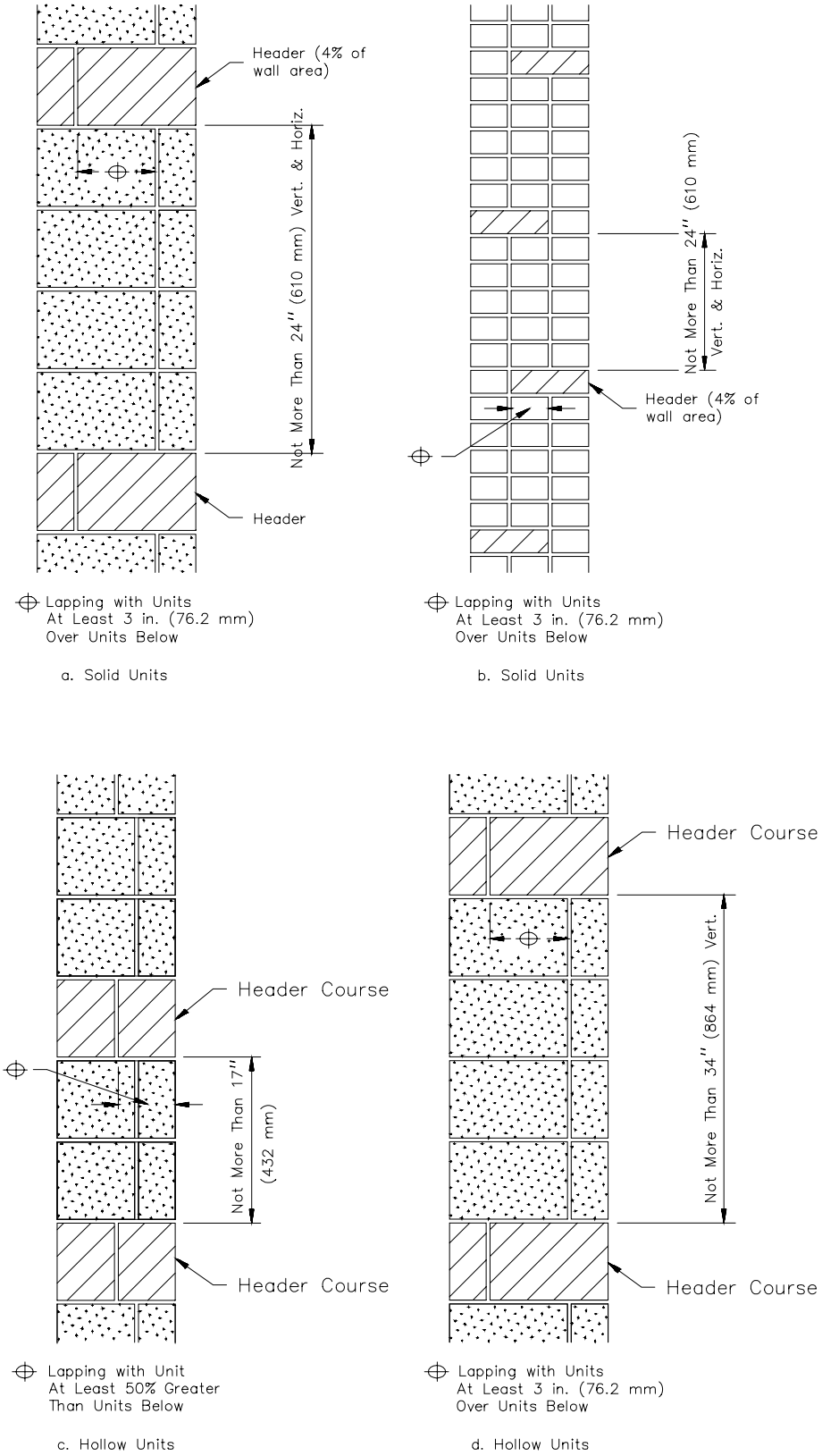
5.2. "Recommended Minimum Requirements for Masonry Wall Construction," *Publication* No. BH6, National Bureau of Standards, Washington, DC, 1924.

5.3. "Modifications in Recommended Minimum Requirements for Masonry Wall Construction," National Bureau of Standards, Washington, DC, 1931.

5.4. "American Standard Building Code Requirements for Masonry," (ASA A 41.1), American Standards Association, New York, NY, 1944.

5.5. "American Standard Building Code Requirements for Masonry," (ANSI A 41.1), American National Standards Institute, New York, NY, 1953 (1970).

5.6 "Standard Specifications and Load Tables for Steel Joists and Joist Girders", Steel Joist Institute, Myrtle Beach, SC, 2002.



~~Fig.~~ Figure 5.7-1 — Cross section of wall elevations

CHAPTER 6
VENEER

6.1 — General

6.1.1 *Scope*

Adhered and anchored veneer definitions given in Section 1.6 are straightforward adaptations of existing definitions. See Figures. 6.1-1 and 6.1-2 for typical examples of anchored and adhered veneer, respectively.

The traditional definition of veneer as an element without resistance to imposed load is adopted. The definition given is a variation of that used in model building codes. Modifications have been made to the definitions to clearly state how the veneer is handled in design.

The design of the backing should be in compliance with the appropriate standard for that material. Suggested standards are:

concrete ... ACI 318, Building Code Requirements for Reinforced Concrete[6.1], American Concrete Institute

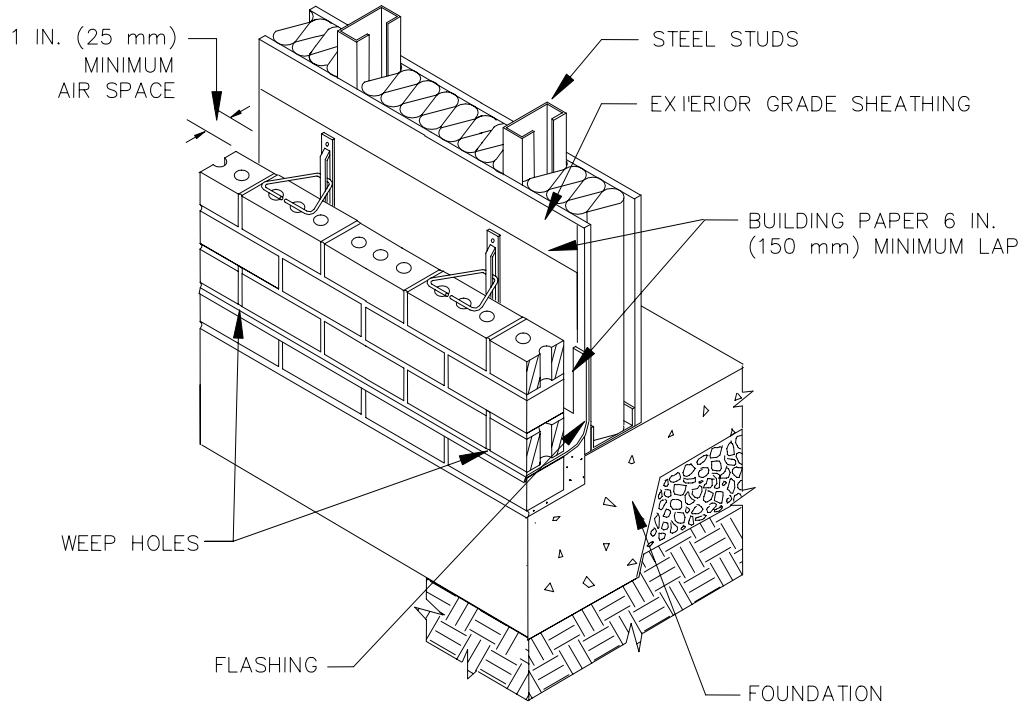
masonry .. Chapters 1 through 5 of this Code

steel..... Design for Cold-formed Steel Structural Members[6.2], American Iron and Steel Institute

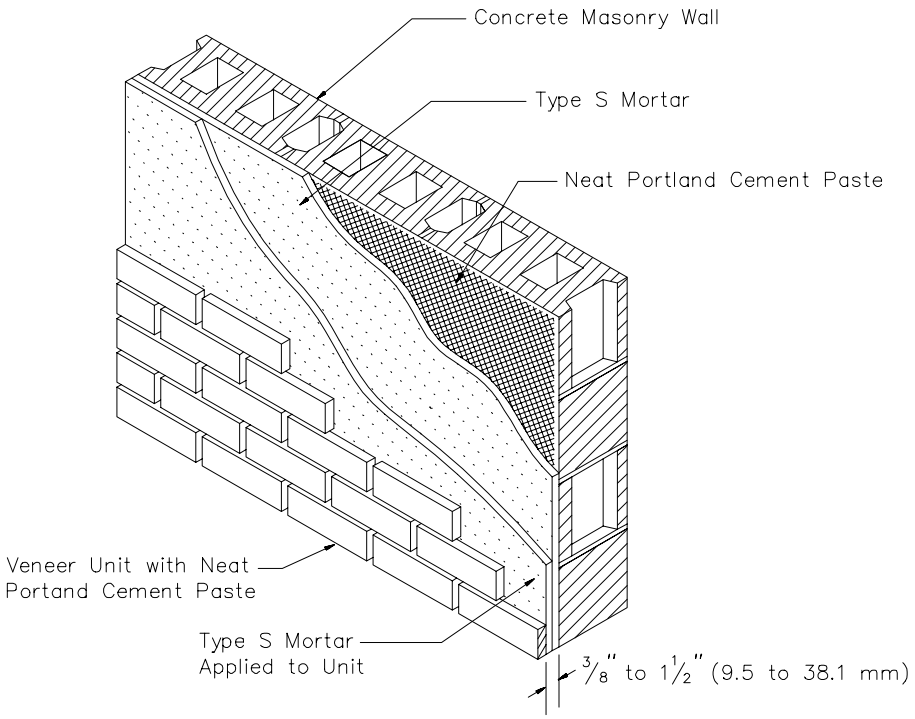
wood ..... National Design Specification for Wood Construction[6.3], American Forest and Paper Association

6.1.1.1 Since there is no consideration of stress in the veneer, there is no need to specify the compressive strength of masonry.

6.1.1.3 The Specification was written for construction of masonry subjected to design stresses in accordance with the other chapters of this Code. Masonry veneer, as defined by this Code, is not subject to those design provisions. The Specification articles that are excluded cover materials and requirements that are not applicable to veneer construction or are items covered by specific requirements in this Chapter and are put here to be inclusive.



~~Fig.~~*Figure 6.1-1 — Anchored veneer*



~~Fig.~~*Figure 6.1-2 — Adhered veneer*

6.1.2 *Design of anchored veneer*

Implicit within these requirements is the knowledge that the veneer transfers out-of-plane loads through the veneer anchors to the backing. The backing accepts and resists ~~all~~ the anchor loads and is designed to resist ~~all~~ the out-of-plane loads.

When utilizing anchored masonry veneer, the designer should consider the following conditions and assumptions:

a) The veneer may crack in flexure under service load.
b) Deflection of the backing should be limited to control crack width in the veneer and to provide veneer stability.
c) Connections of the anchor 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.
f) Requirements for corrosion protection and fire resistance must be included.

If the backing is masonry and the exterior masonry wythe is not considered to add to the out-of-plane load resisting performance of the wall, the exterior wythe is masonry veneer. However, if the exterior wythe is considered to add to the load-resisting performance of the wall, the wall is properly termed a multiwythe, non-composite wall rather than a veneer wall. Such walls are designed under Chapters 2 and 4 of this Code.

Manufacturers of steel studs and sheathing materials have published literature on the design of steel stud backing for anchored masonry veneer. Some recommendations have included composite action between the stud and the sheathing and load carrying participation by the veneer. The Metal Lath/Steel Framing Association has promoted a deflection limit of stud span length divided by 360[6.4]. The Brick Industry Association has held that an appropriate deflection limit should be in the range of stud span length divided by 600 to 720. The deflection is computed assuming that all of the load is resisted by the studs[6.5]. Neither set of assumptions will necessarily ensure that the veneer remains uncracked at service load. In fact, the probability of cracking may be high[6.6]. However,

post-cracking performance is satisfactory if the wall is properly designed, constructed and maintained with appropriate materials[6.7]. Plane frame computer programs are available for the rational structural design of anchored masonry veneer[6.6].

A deflection limit of stud span length divided by 200 ~~times~~ multiplied by the specified veneer thickness provides a maximum uniform crack width for various heights and various veneer thicknesses. Deflection limits do not reflect the actual distribution of load. They are simply a means of obtaining a minimum backing stiffness. The National Concrete Masonry Association provides a design methodology by which the stiffness properties of the masonry veneer and its backing are proportioned to achieve compatibility[6.8].

Masonry veneer with wood frame backing has been used successfully on one- and two-family residential construction for many years. Most of these applications are installed without a deflection analysis.

6.1.3 *Design of adhered veneer*

Adhered veneer differs from anchored veneer in its means of attachment. The designer should consider conditions and assumptions given in Code Section 6.3.1 when designing adhered veneer.

6.1.4 *Dimension stone*

Dimension stone veneer should be covered as a Special System of Construction, under Code Section 1.3.

6.1.5 *General design requirements*

Water penetration through the exterior veneer is expected. The wall system must be designed and constructed to prevent water from entering the building.

The requirements given here and the minimum air space dimensions of Sections 6.2.2.6.3, 6.2.2.7.4, and 6.2.2.8.2 are those required for a drainage wall system. Proper drainage requires weep holes and a clear air space. It may be difficult to keep a 1-in. (25-mm) air space free from mortar bridging. Other options are to provide a wider air space, a vented air space, or to use the rain screen ~~principle.~~

~~Masonry~~principle. 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 and concrete masonry, and address such items as joint reinforcement, slip joints, and sealant joints [6.9, 6.10, 6.11].

Vertical movement joints can be used to accommodate differential movement between vertical bands of dissimilar materials.

6.2 — Anchored veneer

6.2.1 *Alternative design of anchored masonry veneer*

There are no rational design provisions for anchored veneer in any code or standard. The intent of Section 6.2.1 is to permit the designer to use alternative means of supporting and anchoring masonry veneer. See Commentary Section 6.1.1 for conditions and assumptions to consider. The designer may choose to not consider stresses in the veneer or may limit them to a selected value, such as the allowable stresses of Section 2.2, the anticipated cracking stress, or some other limiting condition. The rational analysis used to distribute the loads must be consistent with the assumptions made. See Commentary Section 6.2.2.5 for information on anchors.

The designer should provide support of the veneer; control deflection of the backing; consider anchor loads, stiffness, strength and corrosion; water penetration; and air and vapor transmission.

6.2.2 *Prescriptive requirements for anchored masonry veneer*

The provisions are based on the successful performance of anchored masonry veneer. These have been collected from a variety of sources and reflect current industry practices. Changes result from logical conclusions based on engineering consideration of the backing, anchor, and veneer performance.

6.2.2.3 *Vertical support of anchored masonry veneer* — These requirements are based on current industry practice and current model building codes. Support does not need to occur at the floor level; it can occur at a window head or other convenient location.

The full provisions for preservative-treated wood foundations are given in the National Forest Products Association Technical Report 7[6,9,12].

There are no restrictions on the height limit of veneer backed by masonry or concrete, nor are there any requirements that the veneer weight be carried by intermediate supports. The designer should consider the effects of differential movement on the anchors and connection of the veneer to other building components.

Support of anchored veneer on wood is permitted in previous model building codes. The vertical movement joint between the veneer on different supports reduces the possibility of cracking

due to differential settlement, The height limit of 12 ft (3.7 m) was considered to be the maximum single story height and is considered to be a reasonable fire safety risk.

6.2.2.5 *Anchor requirements* — It could be argued that the device between the veneer and its backing is not an anchor as defined in the Code. That device is often referred to as a tie. However, the term anchor is used because of the widespread use of anchored veneer in model building codes and industry publications, and the desire to differentiate from tie as used in other chapters.

U.S. industry practice has been combined with the requirements of the Canadian Standards Association[6.1-03] to produce the requirements given. Each anchor type has physical requirements that must be met. Minimum embedment requirements have been set for each of the anchor types to ensure load resistance against push-through or pull-out of the mortar joint. Maximum air space dimensions are set in Sections 6.2.2.6 through 6.2.2.8.

There are no performance requirements for veneer anchors in previous codes. Indeed, there are none in the industry. Tests on anchors have been reported[6.4, 6.-11-14]. Many anchor manufacturers have strength and stiffness data for their proprietary anchors.

Veneer anchors typically allow for movement in the plane of the wall but resist movement perpendicular to the veneer. The mechanical play in adjustable anchors and the stiffness of the anchor influence load transfer between the veneer and the backing. Stiff anchors with minimal mechanical play provide more uniform transfer of load, increase the stress in the veneer, and reduce veneer deflection.

The anchors listed in Section 6.2.2.5.6.1 are thought to have lower strength or stiffness than the more rigid plate-type anchors. Thus fewer plate-type anchors are required. These provisions may result in an increase in the number of anchors required when compared to the editions of the BOCA and SBCCI model building codes published in 1993 and 1991, respectively[6.1-25, 6.1-36]. The number of anchors required by this Code ~~decreases in low seismic zones~~is based on~~from~~ the requirements ~~in~~ of the 1991 UBC[6.1-47]. The number of required anchors is increased in the higher Seismic Design Categories. Anchor spacing is independent of backing type.

Anchor frequency should be calculated independently for the wall surface in each plane. That is, horizontal spacing of veneer anchors should not be continued from one plane of the veneer to another.

6.2.2.6 *Masonry veneer anchored to wood backing* — These requirements are similar to those used by industry and given in model building codes for years. The limitation on fastening

corrugated anchors at a maximum distance from the bend is new. It is added to achieve better performance. The maximum distances between the veneer and the sheathing or wood stud is provided in order to obtain minimum compression capacity of anchors.

6.2.2.7 *Masonry veneer anchored to steel backing* — Most of these requirements are new, but they generally follow recommendations in current use[6.5, 6.158]. The minimum base metal thickness is given to provide sufficient pull-out resistance of screws.

6.2.2.8 *Masonry veneer anchored to masonry or concrete backing* — These requirements are similar to those used by industry and have been given in model building codes for many years.

6.2.2.9 *Veneer laid in other than running bond* — Masonry laid in other than running bond has similar requirements in Section 1.11. The area of steel required in Section 6.2.2.9 is equivalent to that in Section 1.11 for a nominal 4-in. (102-mm) wythe.

6.2.2.10 *Requirements in seismic areas* — These requirements provide several cumulative effects to improve veneer performance under seismic load. Many of them are based on similar requirements given in Chapter 30 of the Uniform Building Code[6.147]. 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. An increased number of anchors increases veneer stability and reduces the possibility of falling debris. Joint reinforcement provides ductility and post-cracking strength. Added expansion joints further articulate the veneer, permit greater building deflection without veneer damage and limit stress development in the veneer.

6.2.2.11 *Requirements in areas of high winds* — These reductions are based on the ratio of $(110/130)^2$, the square of the ratio of wind speed in the two locations.

6.3 — Adhered veneer

6.3.1 *Alternative design of adhered masonry veneer*

There are no rational design provisions for adhered veneer in any code or standard. The intent of Section 6.3.1 is to permit the designer to use alternative unit thicknesses and areas for adhered veneer. The designer should provide for adhesion of the units, control curvature of the backing, and consider freeze-thaw cycling, water penetration, and air and vapor transmission. The Tile Council of America limits the deflection of the backing supporting ceramic tiles to span length divided by 360[6.169].

6.3.2 *Prescriptive requirements for adhered masonry veneer*

Similar requirements for adhered veneer have been in the Uniform Building Code[6.1~~47~~] since 1967. The construction requirements for adhered veneer in the Specification have performed successfully[6.~~1720~~].

6.3.2.1 *Unit sizes* — The dimension, area, and weight limits are imposed to reduce the difficulties of handling and installing large units and to assure good bond.

6.3.2.2 *Wall area limitations* — Selecting proper location for movement joints involves many variables. These include: changes in moisture content, inherent movement of materials, temperature exposure, temperature differentials, strength of units, and stiffness of the backing.

6.3.2.3 *Backing* — These surfaces have demonstrated the ability to provide the necessary adhesion when using the construction method described in the Specification. Model building codes contain provisions for metal lath and portland cement plaster. For masonry or concrete backing, it may be desirable to apply metal lath and plaster. Also, refer to ACI 524R, "Guide to Portland Cement Plastering"[6.~~1821~~] for metal lath, accessories, and their installation. These publications also contain recommendations for control of cracking.

6.3.2.4 The required shear strength of 50 psi (345 kPa) is an empirical value based on judgment derived from historical use of adhered veneer systems similar to those permitted by Article 3.3 C of ACI 530.1/ASCE 6/TMS 602. This value is easily obtained with workmanship complying with the Specification. It is anticipated that the 50 psi (345 kPa) will account for differential shear stress between the veneer and its backing in adhered veneer systems permitted by this Code and Specification.

The test method is used to verify shear strength of adhered veneer systems that do not comply with the construction requirements of the Specification or as a quality assurance test for systems that do comply.

References

6.1. Building Code Requirements for Reinforced Concrete, ACI 318-95, American Concrete Institute, Detroit, MI, 1995.

6.2. Specification for the Design of Cold-Formed Steel Structural Members, American Iron and Steel Institute, August 10, 1986 Edition with December 11, 1989 Addendum, American Iron and Steel Institute, Washington, D.C., 1989.

6.3. ANSI/NFoPA National Design Specification for Wood Construction, American Forest & Paper Association, Washington, D.C., 1991.

6.4. Brown, R.H. and Arumula, J.O., "Brick Veneer with Metal Stud Backup - An Experimental and Analytical Study," Proceedings Second North American Masonry Conference, The Masonry Society, Boulder, CO, August 1982, pp. 13-1 to 13-20.

6.5. "Brick Veneer Steel Stud Panel Walls," Technical Notes on Brick Construction No. 28B Revised II, Brick Institute of America, Reston, VA, February 1987.

6.6. Grimm, C.T. and Klingner, R.E., "Crack Probability in Brick Veneer over Steel Studs," Proceedings Fifth North American Masonry Conference, The Masonry Society, Boulder, CO, June 1990, pp. 1323-1334.

6.7. Kelly, T., Goodson, M., Mayes, R., and Asher, J., "Analysis of the Behavior of Anchored Brick Veneer on Metal Stud Systems Subjected to Wind and Earthquake Forces," Proceedings Fifth North American Masonry Conference, The Masonry Society, Boulder, CO, June 1990, pp. 1359-1370.

6.8. "Structural Backup Systems for Concrete Masonry Veneers," NCMA TEK 16-3A, National Concrete Masonry Association, Herndon, VA, 1995.

6.9 NCMA TEK 5-2A: Clay and Concrete Masonry Banding Details, National Concrete Masonry Association, Herndon, VA, 2002.

6.10 BIA E&R Digest on Combinations of Materials, Brick Industry Association, Reston, VA.

6.11 BIA Technical Notes 18A Differential Movement, Design and Detailing of Movement Joints, Part II, December 1991, reissued Sept 2001, Brick Industry Association, Reston, VA, 2001.

6.~~9~~12. "The Permanent Wood Foundation System," Technical Report No. 7, National Forest Products Association (now the American Forest and Paper Association), Washington, DC, January 1987.

6.103. "Connectors for Masonry," CAN3-A370-M84, Canadian Standards Association, Rexdale, Ontario, Canada, 1984.

6.114. "Brick Veneer - New Frame Construction, Existing Frame Construction," Technical Notes on Brick and Tile Construction Number 28, Structural Clay Products Institute (now Brick Institute of America), Reston, VA, August 1966.

6.125. National Building Code, Building Officials and Code Administrators, Country Club Hills, IL, 1993.

6.136. Standard Building Code, Southern Building Code Congress International, Birmingham, AL, 1991.

6.147. Uniform Building Code, International Conference of Building Officials, Whittier, CA, 1991.

6.158. Drysdale, R.G. and Suter, G.T., "Exterior Wall Construction in High-Rise Buildings: Brick Veneer on Concrete, Masonry or Steel Stud Wall System," Canada Mortgage and Housing Corporation, Ottawa, Ontario, Canada, 1991.

6.169 . "Handbook for Ceramic Tile Installation," Tile Council of America, Anderson, SC, January 1996.

6.1720 . Dickey, W.L., "Adhered Veneer in Earthquake, Storm, and Prefabrication," Proceedings, 2nd North American Masonry Conference, College Park, MD, The Masonry Society, Boulder, CO, August 1982.

6.1821. Guide to Portland Cement Plastering, ACI 524R-93, American Concrete Institute, Farmington Hills, MI,1993.

CHAPTER 7
GLASS UNIT MASONRY

7.1 — General

7.1.1 *Scope*

Glass unit masonry is used as a nonload-bearing elements in interior and exterior walls, partitions, and in window openings, and as an architectural features. CodeDesign provisions in the Code are empirical., These provisions are cited inbased on previous codes, are based on successful performance, and are recommended by manufacturers'. recommendations.

7.1.1.1 Since there is no consideration of stress in glass unit masonry, there is no need to specify the compressive strength of masonry.

7.2 — 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. Contact glass block manufacturers for technical data on the fire resistance ratings of panels, or refer to the latest issue of UL Building MaterialsFire Resistance Directory – Volume 3[7.1] and the local building code.

7.2.1 *Exterior standard-unit panels*

The wind load resistance curve[7.2,7.3, 7.5] (Fig.Figure7.2-1) is representative of the ultimate load limits for a variety of panel conditions. Historically, tThea 144-ft^2 (13.37-m^2) area limit is basedhas been referenced in building codes as the maximum area permitted in exterior applications, without reference to any safety factor or design wind pressure. The 144-ft^2 (13.37-m^2) area also reflects the size of panels tested by the National Concrete Masonry Association[7.5]. The 144-ft^2 (13.37-m^2) area limitation provides on a safety factor of 2.7 when the design wind pressure is 20 psf[7.4] (958 Pa).

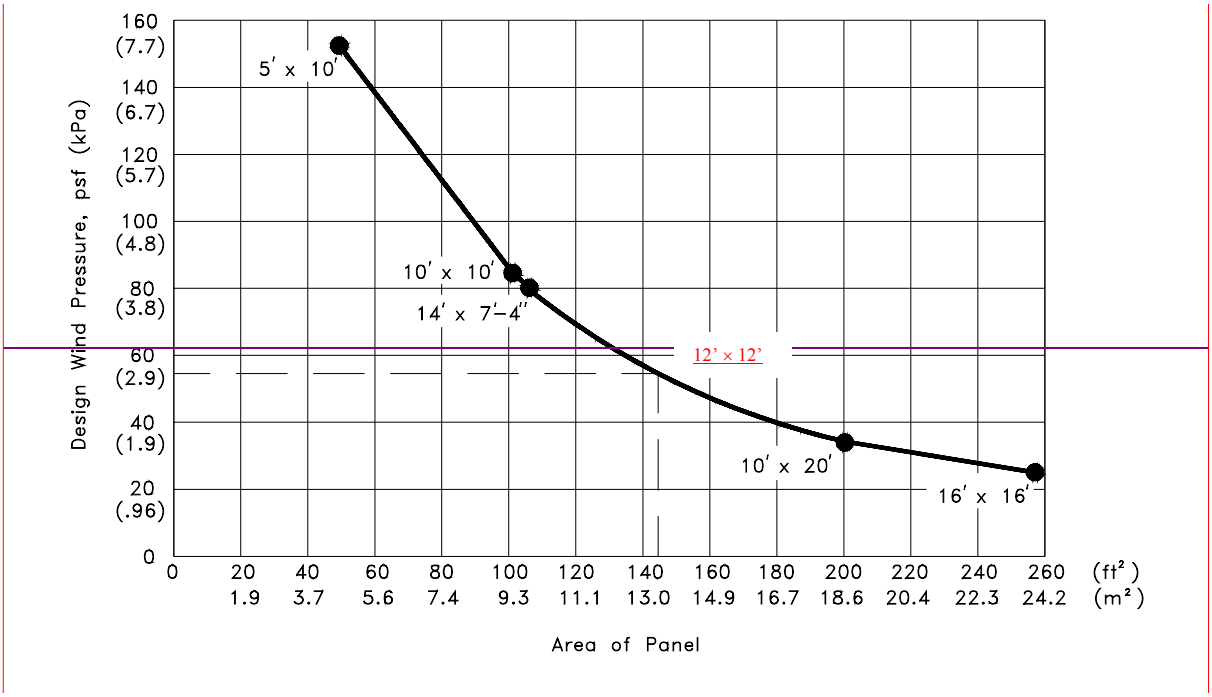
7.2.2 *Exterior thin-unit panels*

There is no historical data for developing a curve for thin units. The Committee recommends limiting the exterior use of thin units to areas where the design wind pressure does not exceed 20 psf (958 Pa).

7.3 —Support

7.3.2 *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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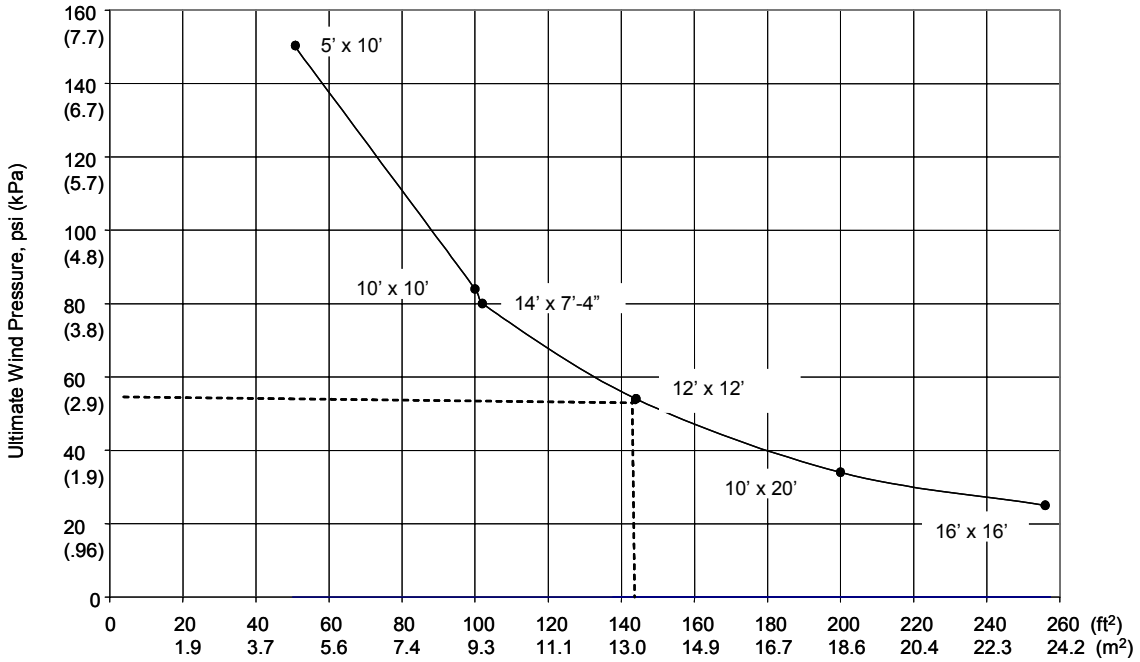

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3 **Example of how to use wind-load resistance curve:** If using a design wind pressure of 20 psf
4 (958 Pa), multiply by a safety factor of 2.7 and locate 54 psf (2586 Pa) wind pressure (on vertical
5 axis), read across to curve and read corresponding 144 -ft^2 (13.37-m^2) maximum area per panel (on
6 horizontal axis).

7 ~~*Fig.*~~ *Figure 7.2-1 — Glass masonry ultimate wind load resistance*

7.3.3 *Lateral*

The Code requires glass unit masonry panels to be laterally supported by panel anchors or channel-type restraints. See Figures. 7.3-1 and 7.3-2 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 elements so long as they provide the required lateral support.

7.5 — Base surface treatment

Current industry practice and recommendations by glass block manufacturers state that ~~all~~ surfaces on which glass unit masonry is placed be coated with an asphalt emulsion[7.2, 7.3]. The asphalt emulsion provides a slip plane at the panel base. This is in addition to the expansion provisions at head and jamb locations. The asphalt emulsion also waterproofs porous panel bases.

Glass unit masonry panels subjected to structural investigation tests by the National Concrete Masonry Association[7.5] to confirm the validity and use of the Glass Unit Masonry Design Wind Load Resistance chart (~~Fig.~~Figure 7.2-1) of the Code, were constructed on bases coated with asphalt emulsion. Asphalt emulsion on glass unit masonry panel bases is needed to be consistent with these tests.

References

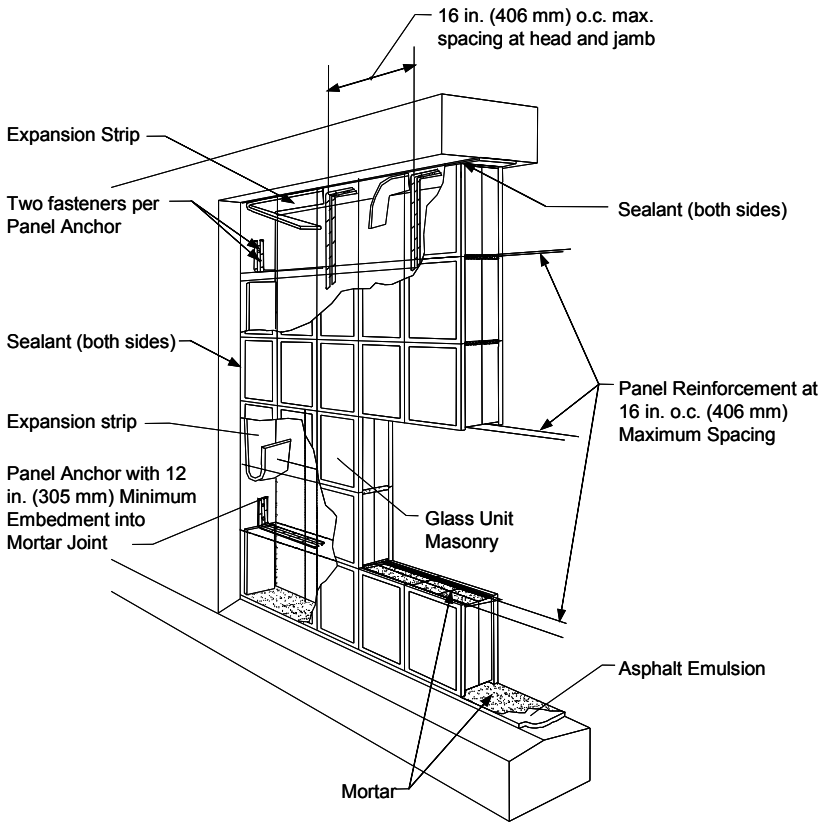
7.1. "~~Building Materials~~Fire Resistance Directory – Volume 3," File No. R2556, Underwriters Laboratories, Inc., Northbrook, IL, 1995.

7.2. "PC Glass Block Products," Installation Brochure (GB-185), Pittsburgh Corning Corp., Pittsburgh, PA, 1992.

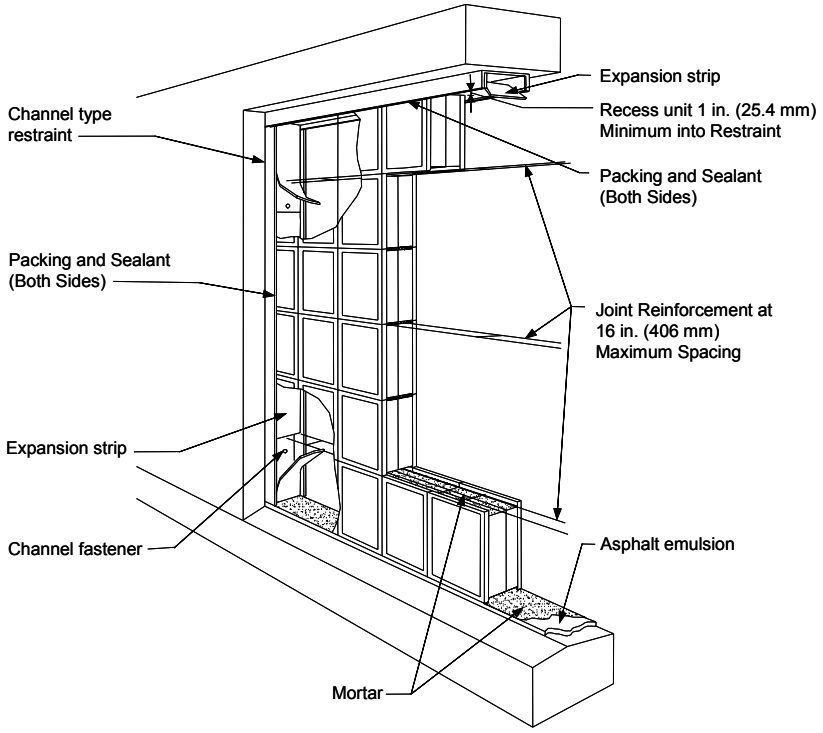
7.3. "WECK Glass Blocks," Glashaus Inc., Arlington Heights, IL, 1992.

7.4. Smolenski, Chester P., "A Study of Mortared PCC Glass Block Panel Lateral Load Resistance (Historical Perspective and Design Implications)," Pittsburgh Corning Corporation, Pittsburgh, PA, 1992.

7.5. Structural Investigation of Pittsburgh Corning Glass Block Masonry, National Concrete Masonry Association Research and Development Laboratory, Herndon, VA, August 1992.



~~Fig.~~ *Figure 7.3-1 — Panel anchor construction*



~~Fig.~~ Figure 7.3-2 — Channel-type restraint construction

APPENDIX A
STRENGTH DESIGN OF AUTOCLAVED AERATED CONCRETE (AAC) MASONRY

A.1 —General

~~**A.1.3** *Design strength*~~

~~In order to accurately distribute loads in an AAC masonry structure subjected to lateral loading, the lateral stiffness of all structural members should be considered. Although structures may be designed to use shear walls for lateral-load resistance, columns may also be incorporated for vertical capacity. The stipulation that lateral load resisting elements provide at least 80 percent of the lateral stiffness helps ensure that additional elements do not significantly contribute to the lateral stiffness. Based on typical design assumptions, the lateral stiffness of structural elements of AAC masonry should be based on cracked section properties for reinforced masonry and uncracked section properties for unreinforced masonry.~~

A.1.4 *Strength of joints*

Design provisions of Appendix A and prescriptive seismic reinforcement requirements of Section ~~1.14~~1.16 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 Eq. (A-12b).

A.1.5 *Strength-reduction factors*

The strength-reduction factor incorporates the difference between the nominal strength provided in accordance with the provisions of Appendix A 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.

A.1.5.1*Combinations of flexure and axial load in 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 elements. The higher strength-reduction factor associated with reinforced elements (in comparison to unreinforced elements) reflects a decrease in the coefficient of variation of the measured strengths of reinforced elements when compared to similarly configured unreinforced elements.

A.1.5.2*Combinations of flexure and axial load in unreinforced AAC 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 elements. The lower strength-reduction factor associated with unreinforced elements (in comparison to reinforced elements) reflects an increase in the coefficient of variation of the measured strengths of unreinforced elements when compared to similarly configured reinforced elements.

A.1.5.3*Shear* — 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 Appendix A 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.

A.1.5.4*Anchor bolts* — Anchor bolts embedded in grout in AAC masonry behave like those addressed in Chapter 3 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.

A.1.5.5*Bearing* — The value of the strength-reduction factor used in bearing assumes that some degradation has occurred within the masonry material.

A.1.6 *Deformation requirements*

A.1.6.1*Drift limits* — 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).

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 since it may give a false impression of the effects in critical stories. However, it is important when considering seismic separation requirements.

There are many reasons for controlling drift; one is to control member inelastic strain. Although the relationship between lateral drift and maximum strain is imprecise, so is the current state of knowledge of what strain limitations should be. Drift limits for AAC masonry are based on performance of AAC shear-wall test specimens[A.1].

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.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 earthquake- resisting elements.~~

~~The designer must keep in mind that the 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 computing building separations, the distance should be sufficient to avoid damaging contact 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) should be followed.~~

~~**A.1.6.2**~~*Deflection of unreinforced (plain) 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 3.2.1.3.

A.1.6.~~3~~2 *Deflection of reinforced AAC masonry* — Values of I_{eff} are typically about one-half of I_g for common solid grouted element configurations. 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 resulting from unfactored loads as shown in the following equation. This equation has successfully been used for estimating the post-cracking flexural stiffness of both concrete and masonry.

$$I_{eff} = I_n\left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] \le I_n \le 0.5I_g$$

A.1.7 *Anchor bolts*

Headed and bent-bar anchor bolts embedded in grout in AAC masonry behave like those

addressed in Chapter 3 and are designed identically. Anchors for use in AAC masonry units are available from a variety of manufacturers.

A.1.8 *Material properties*

A.1.8.1*Compressive strength*

A.1.8.1.1 *Masonry compressive strength* — Research[A.1, A.2, A.3, A.4] has been conducted on structural components of AAC masonry with a compressive strength of 290 to 1,500 psi (2.00 to 10.34 MPa). Design criteria are based on these research results.

A.1.8.1.2 *Grout compressive strength* — Since most empirically derived design equations relate the calculated 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. 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[A.2, A.3, A.4].

A.1.8.2*Masonry splitting tensile strength* — The equation for splitting tensile strength is based on ASTM C1006 tests[A.2, A.4].

A.1.8.3*Masonry modulus of rupture* —The modulus of rupture is based on tests conducted in accordance with ASTM C 78[A.5] on AAC masonry with different compressive strengths[A.2,A.5,A.6]. 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 tests on AAC shear wall specimenstests[A.3,A.4]. Shear wall tests[A.2] 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.

A.1.8.4*Masonry direct shear strength* **—** The equation for direct shear strength is based on shear tests[A.2,A.4,5]. Based on tests by Kingsley et al[A.7], interface shear strength between grout and conventional masonry units varies from 100 to 250 psi. Based on tests by Tanner[A.2], interface shear strength between grout and AAC material had a 5% fractile (lower characteristic) value of 37

psi. Based on Kingsley's work, the value of 37 psi is probably a conservative bound to the actual value; it can safely and appropriately be used for AAC masonry ~~now, and can probably be revised upward if warranted by future testing~~.

A.1.8.5*Coefficient of friction* **—** The coefficient of friction between AAC and AAC was determined based on direct shear tests performed at The University of Texas at Austin. The coefficient of friction between AAC and leveling mortar was determined based on tests on shear walls at The University of Texas at Austin.

A.1.8.6*Reinforcement strength* — Research[3.2] 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~~minimizes the over-strength unintentionally incorporated into a design~~.

A.1.~~10~~9 *Concentrated loads*

~~**A.1.10.1** Masonry laid in running bond is assumed to distribute the axial compressive stress resulting from a concentrated load along the length of wall as described in this Code. Stress is assumed to be transmitted across the head joints only of masonry laid in running bond. Thus, when other than running bond is used, concentrated loads can only be spread across the length of one unit unless a bond beam or other technique is used to distribute the load (Fig. 2.1-14)~~

A.1.~~10~~9.2 ~~When the supporting masonry area is larger on all sides than the bearing area, this Code allows distribution of concentrated loads over a bearing area A_2 larger than A_1, determined as illustrated in Fig. 2.1-15. This is permissible because the confinement of the bearing area by surrounding masonry increases the bearing capacity of the wall in the vicinity of concentrated loads.~~Commentary Section 1.9.6 gives further information.

A.1.~~10~~9.4 *Bearing for simply supported precast floor and roof members on AAC shear walls* — Bearing should be checked wherever floor or roof elements 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 A.1-1.

A.2.4 *Axial tension*

Commentary Section 2.2.4 provides further information.

A.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.

A.3.2 — *Design assumptions*

For AAC, test results indicate that ε_{mu} is 0.003 and the value of the stress ~~of~~ in the equivalent rectangular stress block is 0.85 f'_{AAC} with $a = 0.67c$ ~~$= \beta_1 = 0.67$~~. [A.2, A.3, A.4]

A.3.3 No commentary.

A.3.3.1 *Reinforcing bar size limitations* — Grout spaces may include, but are not limited to, cores, bond beams, and collar joints. At sections containing lap splices, the maximum area of reinforcement specified in the Code may be doubled.

A.3.3.3.1 *Development of tension and compression reinforcement* — 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.

A.3.4— *Design of beams, piers, and columns*

A.3.4.1.2 *Shear strength provided by reinforcement, V_s* — Test results at UT Austin[A.2, A.4] 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 computing V_s.

A.3.4.1.2.1*Nominal masonry shear strength* —This equation was developed based on observed web shear cracking in shear walls tested at the University of Texas at Austin [A.2, A.3] and Hebel AG in Germany[2]. Flexural shear cracking of AAC shear walls was observed in 6 tests at The University of Texas at Austin[A.1, A.2, A.3]. Although flexural shear cracking can be predicted, it does not correspond to a significant decrease in strength or stiffness and, for that reason

design limits are not proposed. Masonry units laid in other than running bond may exhibit discontinuities at head joints. The nominal masonry shear strength for AAC masonry laid in other than 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[2].

A.3.4.1.2.2 *Nominal shear strength provided by diagonal strut*— This mechanism limits the shear strength at large levels of axial load. It was based on test results[A.32], using a diagonal strut width of $0.25l_w$ based on test observations.

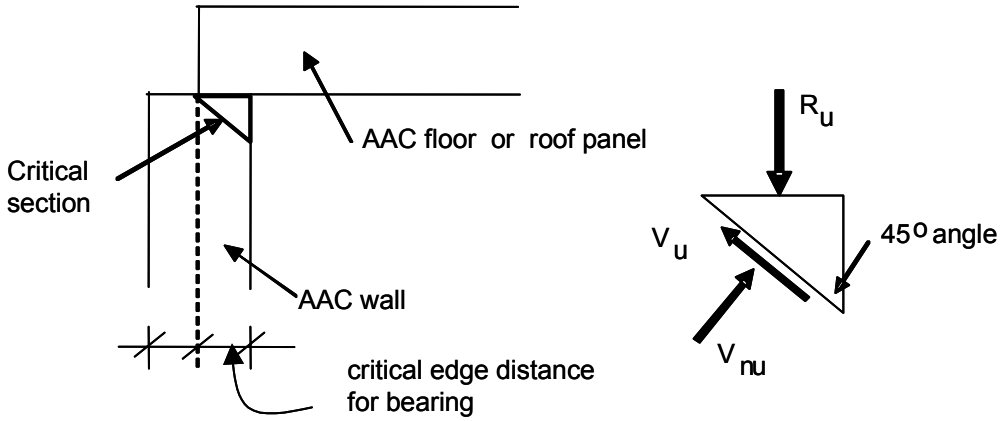


Figure A.1-1 Critical section at bearing of AAC floor or roof panel on AAC wall

A.3.4.1.2.3*Nominal shear strength provided by sliding shear resistance* — This equation was based on test results from the University of Texas at Austin[A.1, A.2, A.3]. Since this failure mode is ductile, the nominal capacity in sliding shear does not need to be considered in the capacity design check of Section A.1.3.

At an unbonded interface, nominal sliding shear capacity should be based on friction only. At an interface where thin-bed mortar is present, the nominal sliding shear capacity should be based on the greater of the capacity based on initial adhesion, and the frictional capacity after that initial adhesion is overcome. At an interface where leveling-bed mortar is present, the interface is probably cracked due to in-plane flexure, and initial adhesion should not be counted on. The nominal sliding shear capacity should be based on the frictional capacity consistent with the total force on the compressive stress block, including the compressive force required to equilibrate the tensile force in the longitudinal reinforcement.

A.3.4.1.2.4 Equation 3-22 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 efficiency factor is the non-uniform distribution of tensile strain in the horizontal reinforcement over the height of the element. For reasons of constructability, AAC walls are traditionally reinforced horizontally with deformed steel in grout-filled bond beams. While no tests have been performed with AAC masonry walls having deformed horizontal reinforcement in concrete bond beams, to use the efficiency factor of 0.5 would be inconsistent with current provisions for reinforced concrete, which do not use the factor. Also, including the efficiency factor would require the use of more shear reinforcement, which in the case of AAC shear walls would increase the probability of brittle failure of the diagonal compression strut.

A.3.5 *Wall design for out-of-plane loads*

A.3.5.3 *Moment and deflection calculations* — 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 A.3.5.

A.3.5.4 *Walls with factored axial stress of 0.20 f'_{AAC} or less* — For slenderness ratios greater than 30, there is an additional limitation on the axial stress. There are currently no strength design provisions for axial stress greater than 0.20 f'_{AAC}. The required moment due to lateral loads, eccentricity of axial load, and lateral deformations are 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 Eq. (A-17). The design formulas provide procedures for determining the nominal moment strength. These formulas take into account the effect of vertical loads increasing the capacity of the section.

A.3.6— *Wall design for in-plane loads*

A.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 3.3.6.5.

A.3.6.7 See Commentary Section 3.3.6.7.

A.3.6.8 See Commentary Section 3.3.6.8.

A.3.6.9 See Commentary Section 3.3.6.9.

A.3.6.10 See Commentary Section 3.3.6.10.

References

A.1. Varela, J. L., Tanner, J. E. and Klingner, R. E., "Development of Seismic Force-Reduction and Displacement Amplification Factors for AAC Structures," *EERI Spectra*, vol. 22, no. 1, February 2006, pp. 267-286.~~Varela, Jorge L., "Development of *R* and C_d Factors for the Seismic Design of AAC Structures," University of Texas at Austin, Dissertation, May 2003~~.

A.2. Tanner, J.E., Varela, J.L., Klingner, R.E., Brightman M. J. and Cancino, U., "Seismic Testing of Autoclaved Aerated Concrete (AAC) Shear Walls: A Comprehensive Review," *Structures Journal*, American Concrete Institute, Farmington Hills, Michigan, vol. 102, no. 3, May - June 2005, pp. 374-382.~~Brightman, Matthew, "AAC Shear Wall Specimens: Development of Test Setup and Preliminary Results," MS Thesis, Department of Civil Engineering, The University of Texas at Austin, May 2000.~~

A.3. Tanner, J.E., Varela, J.L., Klingner, R.E., "Design and Seismic Testing of a Two-story Full-scale Autoclaved Aerated Concrete (AAC) Assemblage Specimen," *Structures Journal*, American Concrete Institute, Farmington Hills, Michigan, vol. 102, no. 1, January - February 2005, pp. 114-119.~~Tanner, Jennifer E., "Design Provisions for Autoclaved Aerated Concrete (AAC) Structural Systems," PhD Dissertation, Department of Civil Engineering, The University of Texas at Austin, May 2003.~~

A.4 Argudo, Jaime, "Evaluation and Synthesis of Experimental Data for Autoclaved Aerated Concrete," MS Thesis, Department of Civil Engineering, The University of Texas at Austin, August 2003.

A.4 ASTM C 78-02 *Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)*, American Society for Materials and Testing, West Conshohocken, PA.

A.6 Fouad, Fouad; Dembowski, Joel; Newman, David, "Material Properties and Structural Behavior of Plain and Reinforced Components," Department of Civil and Environmental Engineering at The University of Alabama at Birmingham, February 28, 2002.

~~A.4 ASTM C 78-02 *Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)*, American Society for Materials and Testing, West Conshohocken, PA.~~

~~A.45 Argudo, Jaime, "Evaluation and Synthesis of Experimental Data for Autoclaved Aerated Concrete," MS Thesis, Department of Civil Engineering, The University of Texas at Austin, August 2003.~~

~~A.5~~ 6 ~~Fouad, Fouad; Dembowski, Joel; Newman, David, "Material Properties and Structural Behavior of Plain and Reinforced Components," Department of Civil and Environmental Engineering at The University of Alabama at Birmingham, February 28, 2002.~~

A.7 ~~A.6 ASTM C 78-02 *Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)*, American Society for Materials and Testing, West Conshohocken, PA.~~ Kingsley, G. R., Tulin, L. G. and Noland, J. L., "The Influence of Water Content and Unit Absorption Properties on Grout Compressive Strength and Bond Strength in Hollow Clay Unit Masonry," Proceedings, Third North American Masonry Conference, Arlington, Texas, 1985.

~~A.7~~8 ~~Klingner, R. E., Tanner, J. E. and Varela, J. L., "Development of Seismic Design Provisions for AAC Structures: An Overall Strategy for the US," *Proceedings*, 9th North American Masonry Conference, Clemson, South Carolina, June 1-4, 2003.~~

~~A.8~~9 ~~Tanner, J. E., Varela, J. L. and Klingner, R. E., "Seismic Testing of AAC Shear Walls: Technical Basis for Proposed Design Provisions," *Proceedings*, 9th North American Masonry Conference, Clemson, South Carolina, June 1-4, 2003.~~

~~A.9~~10 ~~Tanner, J. E., Varela, J. L. and Klingner, R. E., "Seismic Performance of a Two-story AAC Assemblage," *Proceedings*, 9th North American Masonry Conference, Clemson, South Carolina, June 1-4, 2003.~~

~~A.10~~11 ~~Varela, J. L., Tanner, J. E. and Klingner, R. E., "Development of *R* and C_d Factors for Seismic Design of AAC Structures," *Proceedings*, 9th North American Masonry Conference, Clemson, South Carolina, June 1-4, 2003.~~

~~A.11~~12 ~~Tanner, J. E., Varela, J. L., Brightman, M. T., Cancino, U. and Klingner, R. E., "Seismic Performance and Design of Autoclaved Aerated Concrete Structural Systems," *Proceedings*, 13th World Conference in Earthquake Engineering, Vancouver, Canada, August 1-6, 2004. (accepted for publication).~~

~~A.12~~13 ~~Varela, J. L., Tanner, J. E. and Klingner, R. E., Development of Seismic Force and Displacement Modification Factors for Design of AAC Structures," *Proceedings*, 13th World Conference in Earthquake Engineering, Vancouver, Canada, August 1-6, 2004.~~

~~A.13~~14 ~~Klingner, R. E., Tanner, J. E., Varela, J. L., Brightman, M., Argudo, J. and Cancino, U., "Technical Justification for Proposed Design Provisions for AAC Structures: Introduction and Shear Wall Tests," ACI 523A Special Publication, August 2003 (accepted for publication).~~

~~A.14~~15 ~~Klingner, R. E., Tanner, J. E. and Varela, J. L., "Technical Justification for Proposed~~

Design Provisions for AAC Structures: Assemblage Test and Development of R and C_d Factors," ACI 523A Special Publication, August 2003 (accepted for publication).

A.1516 Tanner, J. E., Varela, J. L., Klingner, R. E., Brightman M. J. and Cancino, U., "Seismic Testing of Autoclaved Aerated Concrete (AAC) Shear Walls: A Comprehensive Review," *Structures Journal*, American Concrete Institute, Farmington Hills, Michigan (accepted for publication, September 2003).

A.1617 Tanner, J. E., Varela, J. L., Klingner, R. E., "Design and Seismic Testing of a Two-story Full-scale Autoclaved Aerated Concrete (AAC) Assemblage Specimen," *Structures Journal*, American Concrete Institute, Farmington Hills, Michigan (accepted for publication, September 2003).

A.1718 Varela, J. L., Tanner, J. E. and Klingner, R. E., "Development of Seismic Force-Reduction and Displacement Amplification Factors for AAC Structures." Masonry Buildings with Flexible Roof Diaphragms, Part I: Seismic and Quasi-static Testing," *EERI Spectra* (accepted for publication, September 2003).

Commentary on Specification for Masonry Structures (ACI 530.1-05-xx/ASCE 6-05-xx/TMS 602-05-xx) Reported by the Masonry Standards Joint Committee (MSJC)

INTRODUCTION

Chapter 1 of the *Building Code Requirements for Masonry Structures* (ACI 530-05-xx/ASCE 5-05-xx/TMS 402-05-xx) makes the *Specification for Masonry Structures* (ACI 530.1-05-xx/ASCE 6-05-xx/TMS 602-05-xx) an integral part of the Code. ACI 530.1-05-xx/ASCE 6-05-xx/TMS 602-05-xx Specification 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 covers some of the points that the Masonry Standards Joint Committee (MSJC) considered in developing the provisions of the Code, which are written into this Specification. Further explanation and documentation of some of the provisions of this Specification are included. Comments on specific provisions are made under the corresponding part or section and article numbers of this Code and Specification.

As stated in the Preface, Specification ACI 530.1-05-xx/ASCE 6-05-xx/TMS 602-05-xx is a reference standard which the Architect/Engineer may cite in the contract documents for any project. Owners, through their representatives (Architect/Engineer), may write requirements into contract documents that are more stringent than those of ACI 530.1-05-xx/ASCE 6-05-xx/TMS 602-05-xx. This can be accomplished with supplemental specifications to this Specification.

The contractor should not be required through contract documents to comply with the Code or to assume responsibility regarding design (Code) requirements. The Code 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 function and use of this Specification. 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. ~~All~~ Each project~~s~~ require~~s~~ response to the mandatory requirements.

PART 1 — GENERAL

1.1 — Summary

1.1 C The scope of the work is outlined in this article. All of these tasks and materials will not appear in every project.

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. Definitions have been coordinated between the Code and Specification.

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. The thickness or width is given first, followed by height and length. Nominal dimensions are normally given in whole numbers nearest to the specified dimensions. Specified dimensions are most often used for design calculations.

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 not covered by existing standards, the committee generated its own requirements. Specific dates are given since 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
38800 Country Club Drive
Farmington Hills, MI 48331

www.aci-int.org

American National Standards Institute
25 West 43rd Street,
New York, NY 10036
www.ansi.org

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

American Welding Society
550 N.W. LeJeune Road
Miami, Florida 33126
www.aws.org

Federal Test Method Standard from:
U.S. Army General Material and Parts Center
Petroleum Field Office (East)
New Cumberland Army Depot
New Cumberland, PA 17070

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*

1.4 B.1 There are two separate methods to determine compressive strength of masonry. The unit strength method eliminates the expense of prism tests but is more conservative than the prism

test method. The unit strength method was generated by using prism test data as shown in Figures. 1 and 2. When the method is not specified by the architect or engineer, the Specification permits the contractor to select the method of determining the compressive strength of masonry.

1.4 B.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 Specification Article 1.4 A and Code Section 1.15.7.1.

1.4 B.2.a *Clay masonry* — The values of net area compressive strength of clay masonry in Table 1 were derived using the following equation taken from Reference 1.1:

$$f'_m = A(400 + B f_u)$$

where

A = 1 (inspected masonry)

B = 0.2 for Type N portland cement-lime mortar, 0.25 for Type S or M portland cement-lime mortar

f_u = average compressive strength of ~~brick~~clay masonry units, psi

f'_m = specified compressive strength of masonry

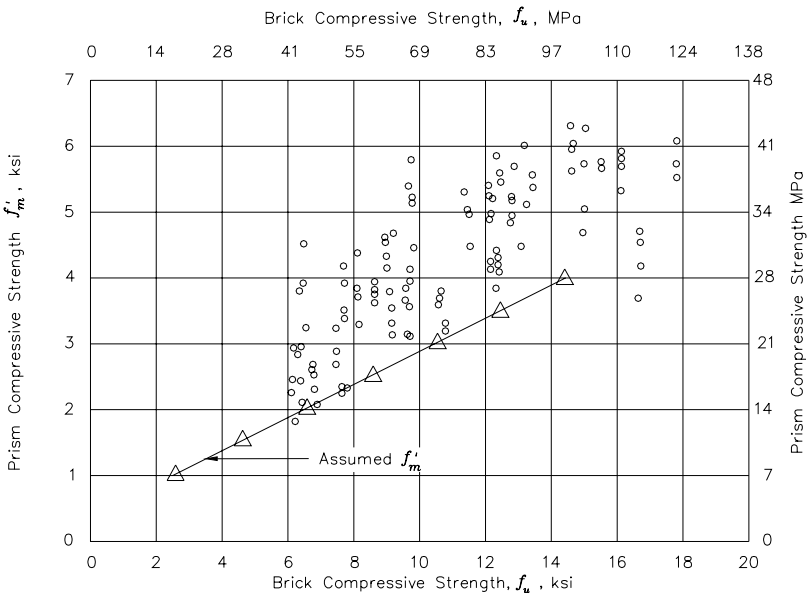
Rearranging terms and letting $A = 1.0$

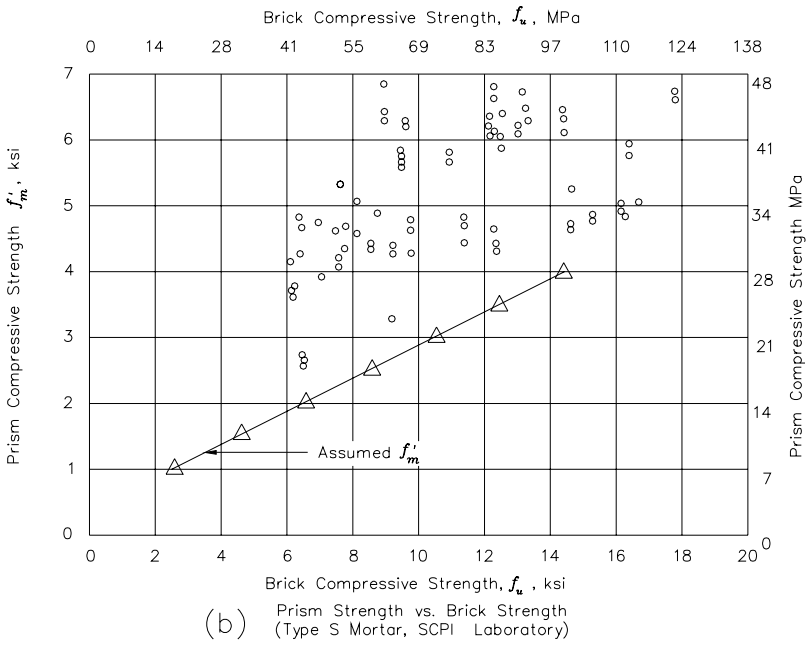
$$f_u = \frac{f'_m - 400}{B}$$

(These equations are for inch-pound units only.)

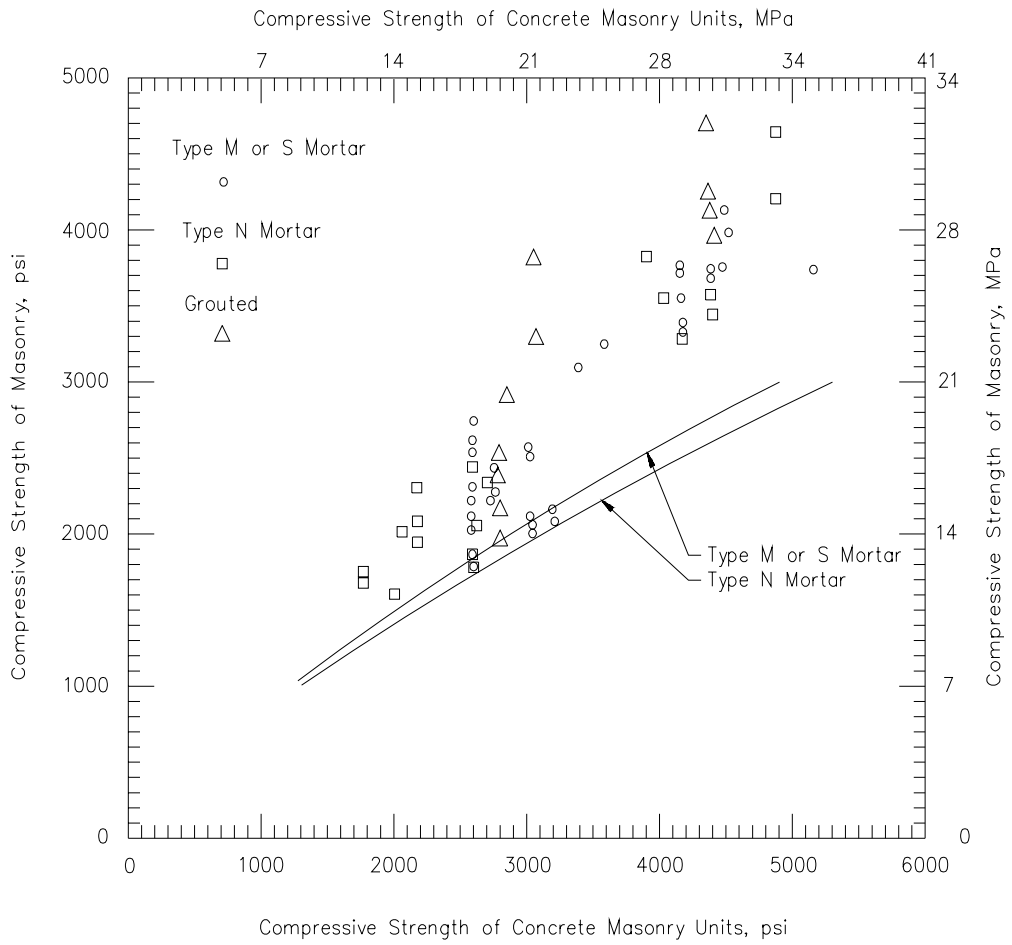
These values were based on testing of solid clay masonry units[1.1] and portland cement-lime mortar. Further testing[1.2] has shown that the values are applicable for hollow clay masonry units and for both types of clay masonry units with all mortar types. A plot of the data is shown in ~~Fig.~~Fig.Figure SC-1.

Reference 1.1 uses a height-to-thickness ratio of five as a basis to establish prism compressive strength. The Code 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 the Code and for all masonry in other codes. The net effect is to increase the net area compressive strength of brick masonry by 22 percent over that in Reference 1.1.





~~Fig.~~~~Fig.~~Figure ~~-~~SC~~-~~1 — Compressive strength of masonry versus clay masonry unit strength



~~Fig.~~Fig.Figure ~~-SC-~~2 — Compressive strength of concrete masonry versus compressive strength of concrete masonry unit~~s strength~~

1.4 B.2.b *Concrete masonry* — In building codes[1.3, 1.4] prior to the Code, the compressive strength of concrete masonry was based on the net cross-sectional area of the masonry unit, regardless of whether the prism was constructed using full or face shell mortar bedding. Furthermore, in ~~these~~ those previous codes, the designer was required to base axial stress calculations on the net area of the unit regardless of the type of mortar bedding ~~used~~. ~~The~~ This Code, in contrast, computes the strength of masonry based on the minimum cross-sectional area of that masonry. If the masonry is fully grouted, the strength is based on the specified cross-sectional area, including the grouted area; if it is ungrouted but fully bedded, the strength is based on the specified net cross-sectional area; and if it is ungrouted and face-shell bedded only, the strength is based on the specified net area of the face shells only.

According to ASTM C 1314, compliance with the specified strength is now determined using a fully bedded prism either grouted or ungrouted to match the specified construction. While each of these changes makes the Code and this Specification easier to use, these changes required a recalibration of earlier prism test data to account for the differences between the compressive strength of hollow unit prisms with full bedding, and those with face-shell bedding. ~~has developed a standard compressive strength of masonry test procedure based on full mortar bedding of the prism. Strength calculations are based on dividing the maximum load on the prism by the net cross-sectional area of the masonry unit.~~

~~Design of concrete masonry sections is based on net cross-sectional area, which requires the designer to differentiate between the face shell mortar bedded area and the full mortar bedded area. These revisions to the methods of determining prism strength and calculating stresses altered the relationship between the unit compressive strength and the compressive strength of masonry to that listed in Table 2 in this Specification.~~

Table 2 lists compressive strength of masonry as related to concrete masonry unit strength and mortar type. These relationships are plotted in ~~Fig.~~~~Fig.~~Figure –SC-2 along with data from 329 tests[1.5 - 1.11]. The curves in ~~Fig.~~~~Fig.~~Figure –SC-2 are shown to be conservative when masonry strength is based on unit strength and mortar type. In order to use face shell bedded prism data in determining the unit strength to masonry compressive strength relationship used in the Specification, a correlation factor between face shell prisms and full bedded prisms was developed. Based on 125 specimens tested with full mortar bedding and face shell mortar bedding, the correlation factor was determined to be 1.29[1.5 - 1.7,1.12]. The face shell bedded prism strength multiplied by this correlation factor determines the full mortar bedded prism strength which is used in the Code.

1.4 B.2.c – *AAC masonry* – The strength of AAC masonry, f'_{AAC}, is controlled by the strength class of the AAC unit as defined by ASTM C 1386. The strength of the thin-bed mortar and its bond in compression and shear will exceed the strength of the unit.

1.4 B.3 *Prism test method* — The prism test method described in ASTM C 1314 was selected as a uniform method of testing clay masonry and concrete masonry to determine ~~its~~ their compressive strengths. The prism test method is used as an alternative to the unit strength method.

Compliance with the specified compressive strength of masonry can be determined by the prism method instead of the unit strength method. ASTM C 1314 uses the same materials and

workmanship to construct the prisms as those to be used in the structure. References 1.13 through 1.17 discuss prism testing. Many more references on the prism test method parameters and results could be added. The adoption of ASTM C 1314 alleviates most of the concerns stated in the above references. ASTM C 1314 replaced ASTM E 447, which was referenced in editions of the Specification prior to 1999.

1.4 C. *Adhered veneer requirements* — Adhesion should be verified if a form release agent, an applied coating, or a smooth surface is present on the backing.

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, and grout.

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 the 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 tests mentioned in Specification Tables 3, 4, and 5. ~~See also the Commentary for Article 1.4.~~

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. Inspection and testing are important components of the quality assurance program, which is used to meet the objective of quality in construction.

Laboratories that comply with the requirements of ASTM C 1093 are more likely to be familiar with masonry materials and testing. Specifying that the testing agencies comply with the requirements of ASTM C 1093 is suggested.

1.6 B. *Inspection Agency's services and duties* — The Code and this Specification require that ~~all~~ masonry be inspected. The allowable stresses used in the Code are based on the premise that the work will be inspected, and that quality assurance measures will be implemented. Minimum testing and minimum inspection requirements are given in Specification Tables 3, 4, and 5. The Architect/Engineer may increase the amount of testing and inspection required. 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.

1.6 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 deficiencies. ~~Items that influence structural performance are controlled by the dimensional t~~Tolerances listed in~~of~~ Specification Article 3.3G 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. ~~All~~ Each procedure~~s~~, 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. Since it serves as a comparison of the finished work, the sample panel should be maintained until ~~all~~ the work has been accepted. 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.

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 jobsite.

Reinforcement and metal accessories are less prone to problems from handling than masonry materials.

1.8 — Project conditions

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 masonry industry recommendations[1.18].

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 jobsite 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.

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, ~~strongly indicates~~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.

1.8 D. *Hot weather construction* — High temperature and low relative humidity increase the rate of moisture evaporation. ~~As temperature increases, the relative humidity at the masonry surface decreases and the evaporation rate increases~~. These conditions can lead to "dryout" (drying of the mortar or grout before sufficient hydration has taken place) of the mortar and grout.[1.19] 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[1.20 - 1.22]. More stringent and extensive hot weather practices may be prudent where temperatures are high, winds are strong, and humidity is low.

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 Specification Commentary 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. ~~Retempering with~~Use of cool water to retemper, when tempering is permitted, restores plasticity and reduces the mortar temperature[1.18, 1.20, 1.21].

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[1.23].

References

1.1. “Recommended Practice for Engineered Brick Masonry,” Brick Institute of America (formerly Structural Clay Products Association), Reston, VA, 1969.

1.2. Brown, R.H., and Borchelt, J.G., “Compression Tests of Hollow Brick Units and Prisms,” *Masonry Components to Assemblages*, ASTM STP 1063, J.H. Matthys, editor, American Society for Testing and Materials, Philadelphia, PA, 1990, pp. 263 - 278.

1.3. ACI Committee 531, Building Code Requirements for Concrete Masonry Structures (ACI 531-79) (Revised 1983)," American Concrete Institute, Detroit, MI, 1983, 20 pp.

1.4. “Specification for the Design and Construction of Load Bearing Concrete Masonry,” (TR-75B), National Concrete Masonry Association, Herndon, VA, 1976.

1.5. Redmond, T.B., “Compressive Strength of Load Bearing Concrete Masonry Prisms,” National Concrete Masonry Association Laboratory Tests, Herndon, VA, 1970, Unpublished.

1.6. Nacos, C.J., “Comparison of Fully Bedded and Face-Shell Bedded Concrete Block,” *Report* No. CE-495, Colorado State University, Fort Collins, CO, 1980, Appendix p. A-3.

1.7. Maurenbrecher, A.H.P., “Effect of Test Procedures on Compressive Strength of Masonry Prisms,” *Proceedings*, 2nd Canadian Masonry Symposium, Carleton University, Ottawa, June 1980, pp. 119-132.

1.8. Self, M.W., “Structural Properties of Loading Bearing Concrete Masonry,” *Masonry: Past and Present*, STP-589, ASTM, Philadelphia, PA, 1975, Table 8, p. 245.

1.9. Baussan, R., and Meyer, C., “Concrete Block Masonry Test Program,” Columbia University, New York, NY, 1985.

1.10. Seaman, J.C., “Investigation of the Structural Properties of Reinforced Concrete Masonry,” National Concrete Masonry Association, Herndon, VA, 1955.

1.11. Hamid, A.A., Drysdale, R.G., and Heidebrecht, A.C., “Effect of Grouting on the Strength Characteristics of Concrete Block Masonry,” *Proceedings*, North American Masonry Conference, University of Colorado, Boulder, CO, Aug. 1978, pp. 11-1 through 11-17.

1.12. Hatzinikolas, M., Longworth, J., and Warwaruk, J., “The Effect of Joint Reinforcement on Vertical Load Carrying Capacity of Hollow Concrete Block Masonry,” *Proceedings*, North American Masonry Conference, University of Colorado, Boulder, CO, Aug. 1978.

1.13. Atkinson, R.H., and Kingsley, G.R., “A Comparison of the Behavior of Clay and Concrete Masonry in Compression,” Atkinson-Noland & Associates, Inc., Boulder, CO, Sept. 1985.

1.14. Priestley, M.J.N., and Elder, D.M., “Stress-Strain Curves for Unconfined and Confined Concrete Masonry,” ACI JOURNAL, *Proceedings* V. 80, No. 3, Detroit, MI, May-June 1983, pp. 192-201.

1.15. Miller, D.E.; Noland, J.L.; and Feng, C.C., “Factors Influencing the Compressive Strength of Hollow Clay Unit Prisms,” *Proceedings*, 5th International Brick Masonry Conference, Washington DC, 1979.

1.16. Noland, J.L., “Proposed Test Method for Determining Compressive Strength of Clay-Unit Prisms,” Atkinson-Noland & Associates, Inc., Boulder, CO, June 1982.

1.17. Hegemier, G.A., Krishnamoorthy, G., Nunn, R.O., and Moorthy, T.V., “Prism Tests for the Compressive Strength of Concrete Masonry,” *Proceedings*, North American Masonry Conference, University of Colorado, Boulder, CO, Aug. 1978, pp. 18-1 through 18-17.

1.18. “Recommended Practices and Guide Specifications for Cold Weather Masonry Construction,” International Masonry Industry All-Weather Council, Washington, DC, 1973.

1.19. Tomasetti, A.A., “Problems and Cures in Masonry” ASTM STP 1063, Masonry Components to Assemblages, ASTM, Philadelphia. PA ,1990, 324-338.

1.20. “All Weather Construction” Technical Notes on Brick Construction Number 1 Revised, Brick Institute of America, Reston, VA, March 1992

1.21. “Hot Weather Masonry Construction,” Trowel Tips, Portland Cement Association, Skokie, IL, 1993

1.22. Panarese, W.C., S.H. Kosmatka, and F.A. Randall Jr “Concrete Masonry Handbook for Architects, Engineers, and Builders,” Portland Cement Association, Skokie, IL, 1991, pp. 121-123.

1.23. “Research Evaluation of Flexural Tensile Strength of Concrete Masonry,” National Concrete Masonry Association, Herndon, VA, 1994.

PART 2 — PRODUCTS

2.1 — Mortar materials

ASTM C 270 contains standards for all 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.

There are two methods of specifying mortar under ASTM C 270: proportions and properties. The proportions specification directs the contractor to mix the materials in the volumetric proportions given in ASTM C 270. These are repeated in Table C-1. The properties specification instructs the contractor to develop a mortar mix that will yield the specified properties under laboratory testing conditions. Table C-2 contains the required results outlined in ASTM C 270. The results are submitted to the owner's representative and the proportions of ingredients as determined in the lab 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 by proportions may have the properties of a different mortar type. A mortar mixed in accordance with the proportion requirements of Table C-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 2. Higher lime content increases workability and water retentivity. ASTM C 270 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 C 270 specifies mortar testing under laboratory conditions only for acceptance of mortar mixes under the property specifications. Field sampling and testing of mortar is conducted under ASTM C 780 and is used to verify consistency of materials and procedures, not mortar strength. ASTM C 1586 provides guidance on appropriate testing of mortar for quality assurance.

2.1 B. *Glass unit masonry* — In exterior applications, certain exposure conditions or panel sizes may warrant the use of mortar type with high bond strength. Type S mortar has a higher bond strength than Type N mortar. Portland cement-lime mortars and mortar-cement mortars have a higher bond strength than some masonry cement mortars of the same type. The ~~specified mortar type should take into account the~~ performance of locally available materials and the size and exposure conditions of the panel should be considered when specifying the type of mortar. Manufacturers of glass units recommend using mortar containing a water-repellent admixture or a cement containing a water-repellent addition.[2.1 – 2.3] A workable, highly water-retentive mortar is recommended ~~during~~ for use when conditions of high heat and low relative humidity ~~conditions~~exist during construction.

2.1 C. *AAC masonry* — ASTM E 72 measures the flexural strength of a full-sized panel, whereas ASTM E 518 and ASTM C 1072 measure the flexural strength of small scale test specimens. ASTM E 72 was developed to provide~~s~~ the most realistic assessment of a wall's performance under flexural loading.

Table C-1 — ASTM C 270 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 2 ¼ 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 C-2 — ASTM C 270 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.

2.2 — Grout materials

ASTM C 476 contains standards for ~~all~~ 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. Since self-consolidating grouts include admixtures and are delivered to the 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 using ASTM C1611/C1611 M.

This article does not apply to prestressing grout; see Article 2.4 G.1.b

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. Since 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.

Concrete brick specified in ASTM C 55 and sand-lime brick specified in ASTM C 73 are specified by grade. ASTM C 55 designates two grades: Grade N and Grade S. Grade N units are for general use, such as in exterior walls above or below grade, which may or may not be exposed to the weather. Grade S units are limited to use above grade in exterior walls with weather-protective coatings and in walls not exposed to weather.

ASTM C 73 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 C-3 summarizes the requirements for various concrete masonry units given in the referenced standards.

ASTM C 744 covers the properties of units that have a resin facing on them. The units must meet the requirements of one of the other referenced standards.

Table C-3 — Concrete masonry unit requirements

ASTM Specification	Unit name	Strength	Weight	Type	Grade
C 55	Concrete brick	yes	yes	yes	yes
C 73	Sand lime brick	yes	no	no	yes
C 90	Load-bearing	yes	yes	yes	no
C 129	Nonload bearing	yes	yes	yes	no
C 744	Prefaced	—	—	—	—

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. Brick is normally exposed in use and most tile is covered. 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 C-4 summarizes the requirements given in the referenced standards.

2.3 C. Stone masonry units are typically selected by color and appearance. The referenced standards classify building stones by the properties shown in Table C-5. The values given in the standards serve as minimum requirements. Stone is often ordered by a particular quarry or 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 cavity. The resulting glass block units are available in a variety of shapes, sizes, and patterns. Underwriters Laboratories inspects the manufacturing and quality control operations of glass block production on a regular basis for UL-approved units. The minimum face thickness is part of that inspection[2.4].

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.

Table C-4 — Clay brick and tile requirements

ASTM Specification	Unit name	Minimum % solid	Grade		
			Strength	Weight	Type
C 34	Load bearing wall tile	a	yes	yes	no
C 56	Nonload bearing wall tile	b	no	yes	no
C 62	Building brick (solid)	75	yes	yes	no
C 126	Ceramic glazed units	c	yes	no	yes
C 212	Structural facing tile	b	yes	no	yes
C 216	Facing brick (solid)	75	yes	yes	yes
C 652	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.

Table C-5 — Stone requirements

ASTM Specification	Stone	Absorption	Density	Compressive strength	Modulus of rupture	Abrasion resistance	Acid resistance
C 503	Marble	minimum	range	minimum	minimum	minimum	none
C 568	Limestone	range	range	range	range	range	none
C 615	Granite	minimum	minimu	minimum	minimum	minimum	none
C 616	Sandstone	range	m	range	range	range	none
C 629	Slate	range	range	none	minimum	minimum	range
			none				

2.3 E. AAC masonry units are specified by both compressive strength and density. Various density ranges are given in ASTM C 1386 for specific compressive strengths. Generally, the density is specified based on considerations of thermal, acoustical, and weight requirements. While ASTM C 1386 provides both ~~a~~ minimum compressive strength and ~~a~~ corresponding average compressive strength values, AAC masonry is structurally designed based on the specific minimum compressive strength of the AAC material as determined by ASTM C 1386.

2.4 — Reinforcement, prestressing tendons, and metal accessories

See Table C-6 for a summary of properties.

2.4 B. *Prestressing tendons* — The constructability 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[2.5]. If mild strength bars (of less than 150 ksi [1034 MPa]) are used, determine the stress-relaxation characteristics by tests and document the results.

2.4 E. *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.

2.4 F. *Coatings for corrosion protection* — Amount of galvanizing required increases with severity of exposure[2.6 – 2.8]. Project documents should specify the level of corrosion protection as required by Code Section 1.15.4.

2.4 G. *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[2.9]. Unit, mortar, and grout cover is not sufficient corrosion protection for bonded prestressing tendons in a corrosive environment. 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 the various corrosion protection systems used for prestressed masonry is available in the literature[2.10]. One example of a corrosion protection system for the prestressing tendon is shown in ~~Fig.~~~~Fig.~~Figure -SC-3.

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.

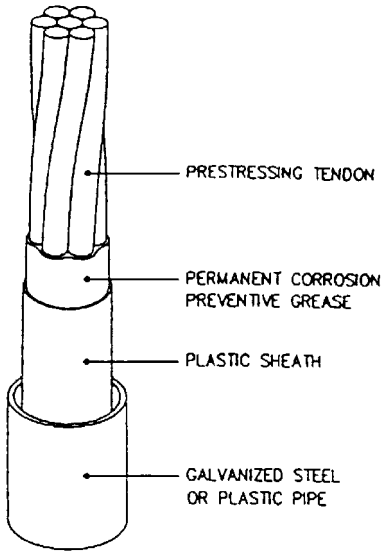
2.4 H. *Prestressing anchorages, couplers, and end blocks* — Typical anchorage and coupling devices are shown in ~~Fig.~~~~Fig.~~Figure -SC-4. Strength of anchorage and coupling devices should be provided by the manufacturer.

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.

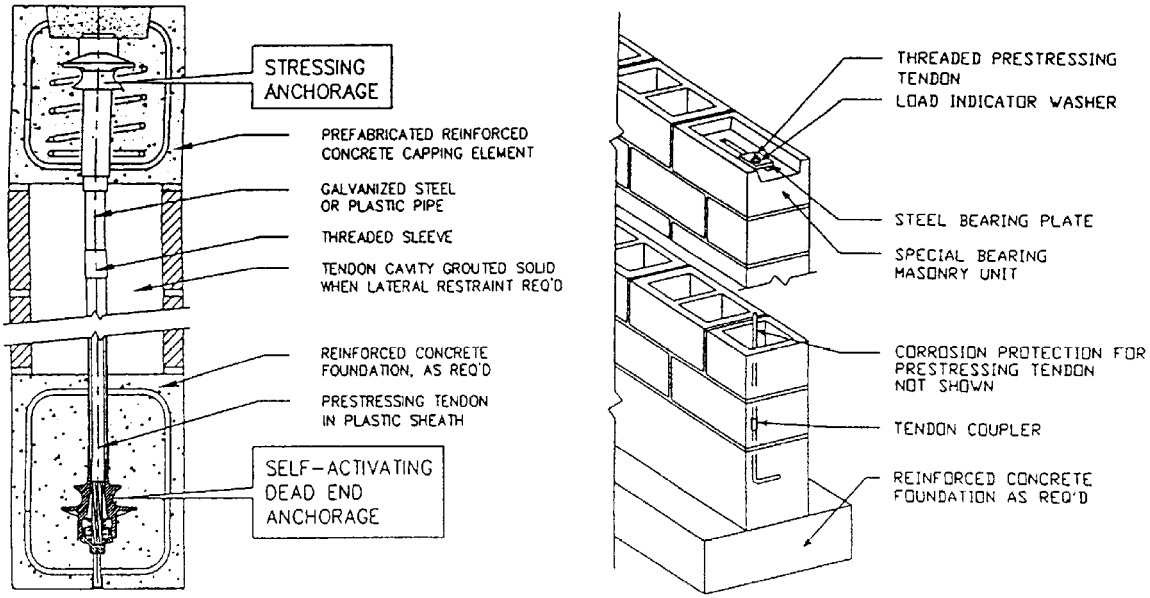
When anchorages and end fittings are exposed, additional precautions to achieve the required fire ratings and mechanical protection for these elements must be taken.

Table C-6 — Reinforcement and metal accessories

ASTM specification	Material	Use	Yield strength, ksi (MPa)	Yield stress, MPa
A 36/A 36M	Structural steel	Connectors	36 (248.2)	250
A 82	Steel wire	Joint reinforcement, ties	70 (482.7)	485
A 167	Stainless steel	Bolts, reinforcement, ties	30 (206.9)	205
A 185	Steel wire	Wire fabric, ties	75 (517.1)	485
A 307	Carbon steel	Connectors	60 (413.7)	—
A 366/A 366M	Carbon steel	Connectors	—	—
A 496	Steel wire	Reinforcement	75 (517.1)	485
A 497	Steel wire fabric	Reinforcement, wire fabric	70 (482.7)	485
A 615/A 615M	Billet steel	Reinforcement	40, 60 (275.8, 413.7)	300, 420
A 996/A 996M	Rail and axle steel	Reinforcement	40, 50, 60 (275.8, 344.8, 413.7)	300, 350, 420
A 706/A 706M	Low alloy steel	Reinforcement	60 (413.7)	—



~~Fig.~~~~Fig.~~Figure -SC-3 — An example of a corrosion protection system for an unbonded tendon



~~Fig.~~Fig.Figure -SC-4 — Typical anchorage and coupling devices for prestressed masonry

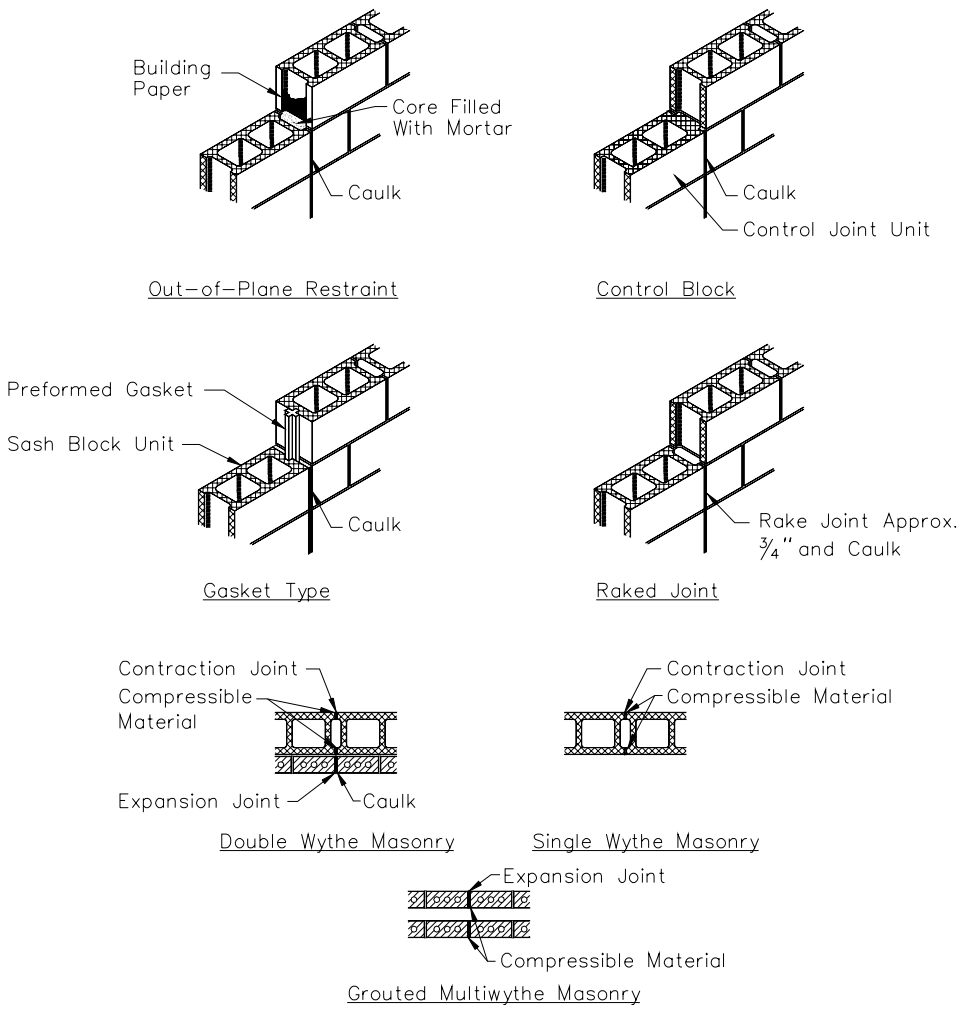
2.5 — Accessories

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[2.11 - 2.13]. Some general rules are: place at returns and jambs of wall openings, maximum spacing of 25 ft (7.62 m) or 3 times wall height. Typical movement joints are illustrated in ~~Fig.~~Fig.Figure -SC-5. 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 C 920 covers elastomeric joint sealants, either single or multi-component. Grade NS, Class 25, Use M is 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.



~~Fig.~~~~Fig.~~Figure ~~SC-~~5 — Movement joints

2.6 — Mixing

2.6 B. *Grout* — ASTM C 476 permits grout to be specified by~~is strictly a~~ proportion or strength requirements~~specification,~~ with fine grout and coarse grout as the two types~~only choices. Their~~. ASTM proportion requirements are given in Table C-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 Specification 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 C 476 has a maximum aggregate size that will pass through a 3/8 in. (9.5 mm) opening. Larger aggregate, conforming to ASTM C 33, can be specified if the grout is placed in areas of unobstructed dimensions greater than 6 in. (152 mm).

Grout meeting the proportion specifications of ASTM C 476 typically has compressive strength ranges shown in Table C-8 when measured by ASTM C 1019. Grout compressive strength is influenced by the water cement ratio, aggregate content, and the type of units used.

Since 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.

Job site proportioning of self-consolidating grout is not permitted since 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 job 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 ~~will~~ exceed the 8 in. to 11 in. (203 to 279 mm) slump range for non self-consolidating grouts.

2.7 — Fabrication

2.7 A. *Reinforcement* — These requirements have been industry standards for ~~a long period of time~~many years.

2.7 B. *Prefabricated masonry* — ASTM C 901 covers the requirements for prefabricated masonry panels, including materials, structural design, dimensions and variations, workmanship, quality control, identification, shop drawings, and handling.

Table C-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 C-8 — Grout strengths

Grout type	Location	Compressive strength, psi (MPa)			Reference
		Low	Mean	High	
Coarse	Lab	1965 (13.55)	3106 (21.41)	4000 (27.58)	2.14
Coarse	Lab	3611 (24.90)	4145 (28.58)	4510 (31.10)	2.15
Coarse	Lab	5060 (34.89)	5455 (37.61)	5940 (40.96)	2.16

References

2.1. “PC Glass Block Products,” (GB 185), Pittsburgh Corning Corp., Pittsburgh, PA, 1992.

2.2. “WECK Glass Blocks,” Glashaus Inc., Arlington Heights, IL, 1992.

2.3. Beall, C., "Tips on Designing, Detailing, and Specifying Glass Block Panels,” The Magazine of Masonry Construction, 3-89, Addison, IL, pp 92 - 99.

2.4. “Follow up Service Procedure,” (File R2556), Underwriters Laboratories, Inc., Northbrook, IL, Ill.1, Sec. 1, Vol. 1.

2.5 Schultz, A.E. and Scolforo, M.J., ‘An Overview of Prestressed Masonry,” *The Masonry Society Journal*, V. 10, No. 1, The Masonry Society, Boulder, CO, August 1991, pp. 6-21.

2.6. Grimm, C.T., “Corrosion of Steel in Brick Masonry,” *Masonry: Research, Application, and Problems*, STP-871, ASTM, Philadelphia, PA, 1985, pp. 67-87.

2.7. Catani, M.J., “Protection of Embedded Steel in Masonry,” *Construction Specifier*, V. 38, No. 1, Construction Specifications Institute, Alexandria, VA, Jan. 1985, p. 62.

2.8. “Steel for Concrete Masonry Reinforcement,” *NCMA TEK* 12-4A, National Concrete Masonry Association, Herndon, VA, 1995, 6 pp.

2.9. “Specifications for Unbonded Single Strand Tendons,” *Post-Tensioning Manual*, 5th Edition, Post-Tensioning Institute, Phoenix, AZ, 1990, pp. 217-229.

2.10. Garrity, S.W., "Corrosion Protection of Prestressing Tendons for Masonry,” *Proceedings, Seventh Canadian Masonry Symposium*, McMaster University, Hamilton, Ontario, June 1995, pp. 736-750.

2.11. Grimm, C.T., "Masonry Cracks: A Review of the Literature,” *Masonry: Materials, Design, Construction, and Maintenance*, STP-992, ASTM, Philadelphia, PA, 1988.

2.12. “Movement,” *Technical Notes on Brick and Tile* No. 18 and 18A, Brick Institute of America, Reston, VA, 1991.

2.13. “Control Joints for Concrete Masonry Walls,” *NCMA TEK* 10-2A, National Concrete Masonry Association, Herndon, VA, 1998, 6 pp.

2.14. ACI-SEASC Task Committee on Slender Walls, "Test Report on Slender Walls,” ACI Southern California Chapter/Structural Engineers Association of Southern California, Los Angeles, CA, 1982, 125 pp.

2.15. Li, D., and Neis, V.V., “The Performance of Reinforced Masonry Beams Subjected to Reversal Cyclic Loadings,” *Proceedings*, 4th Canadian Masonry Symposium, Fredericton, New Brunswick, Canada, June 1986, V. 1, pp. 351-365.

2.16. Unpublished Field Test Report, File 80-617, B'Nai B'Rith Housing, Associated Testing Laboratories, Houston, TX, 1981.

PART 3 — EXECUTION

3.1 — Inspection

3.1 A. The tolerances in this Article are taken from Reference 3.1. The dimensional tolerances of the supporting concrete are important since they control such aspects as mortar joint thickness and bearing area dimensions, which influence the performance of the masonry. Tolerances for variation in grade or elevation are shown in ~~Fig.~~~~Fig.~~Figure –SC-6. 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.

3.2 — Preparation

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 clean space to receive the grout is necessary for this continuity. 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 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.

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 — Masonry erection

Article 3.3 B applies to masonry construction in which the units support their own weight. Face shell mortar bedding of hollow units is standard, except in locations detailed in Specification

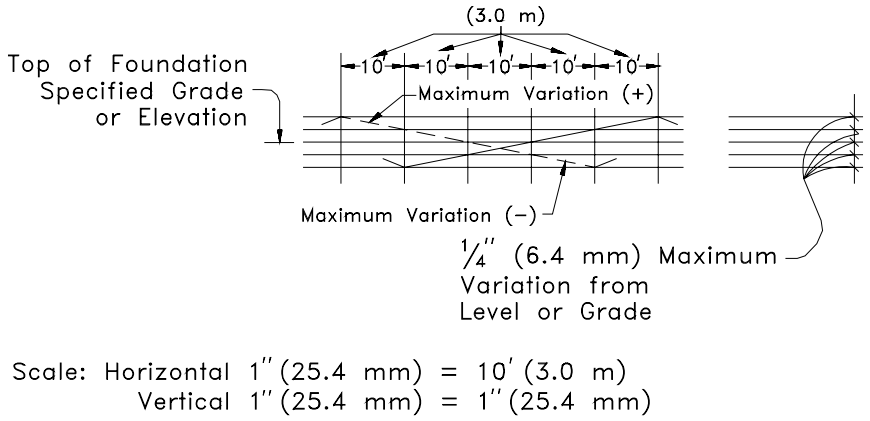
3.3 B.3.b. If full mortar beds throughout are required for structural capacity, for example, the specifier must so stipulate in the project documents.

Article 3.3 C applies to adhered veneer in which the backing supports the weight of the units. This basic method has served satisfactorily since the early 1950s. Properly filled and tooled joints (3.3 C.4) are essential for proper performance of adhered veneer.

Tolerances are established to limit eccentricity of applied load and load carrying capacity of the masonry construction. Since masonry is usually used as an exposed material, it is subjected to tighter dimensional tolerances than those for structural frames. The tolerances given are based on structural performance, not aesthetics.

The provisions for cavity width shown are for the space between wythes of non-composite masonry. The provisions do not apply to situations where masonry extends past floor slabs, spandrel beams, or other structural elements.

The remaining provisions set the standard for quality of workmanship and ensure that the structure is not overloaded during construction.



~~Fig.~~ ~~Fig.~~Figure -SC-6 — Tolerance for variation in grade or elevation

3.3.B.7*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. Since 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.

3.3F - For guidance on bracing of masonry walls for wind, consult Standard Practice for Bracing Masonry Walls Under Construction[3.3].

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.

Tolerances for placement of reinforcement in masonry first appeared in the 1985 Uniform Building Code[3.2]. Reinforcement location obviously influences structural performance of the member. ~~Fig.~~~~Fig.~~Figure -SC-7 illustrates several devices used to secure reinforcement.

3.4 B.6 *Noncontact lap splices* — Lap splices may ~~occur~~be constructed with the bars in adjacent grouted cells if the requirements of this section are met.

3.4 B.8 (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-8) [3.4~~3~~, 3.5~~4~~]. ~~Chipping away~~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.

3.4 C. *Wall ties* — The Code 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 ~~Fig.~~~~Fig.~~Figure -SC-~~8~~9.

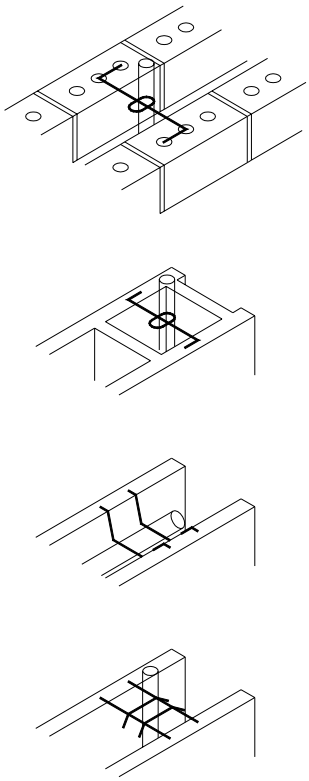


Fig.Fig.Figure -SC-7 — Typical reinforcing bar positioners

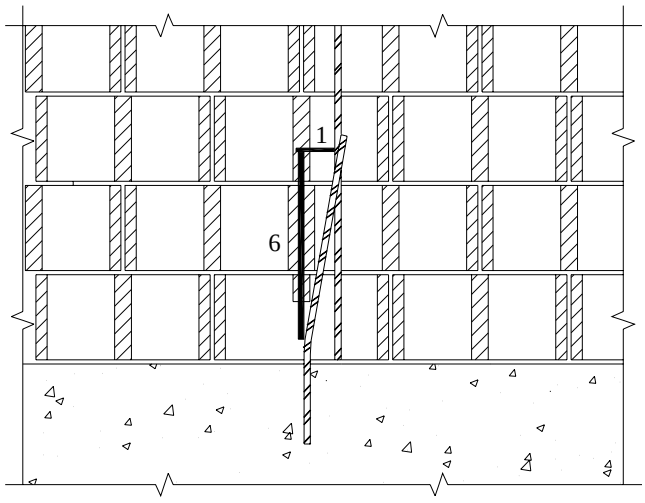
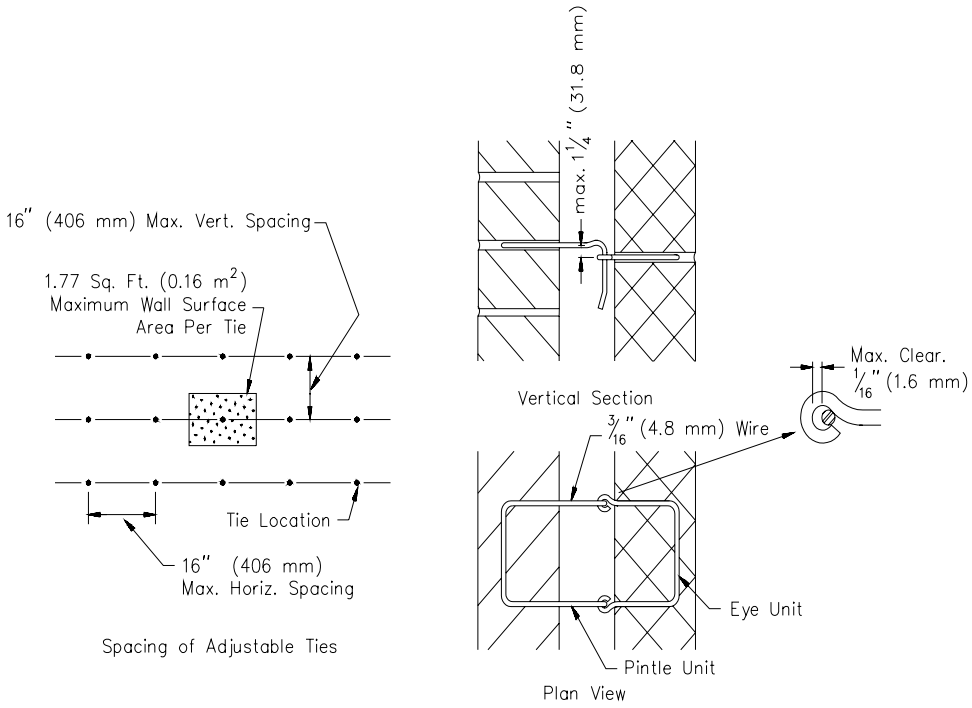


Figure SC-8 – Permitted Bending of Foundation Dowels



~~Fig.~~Fig.~~Fig.~~Figure ~~SC-~~8~~-~~9 — Adjustable ties

3.4 ~~D~~E. *Veneer anchors* — 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.

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 ~~will~~ influence the choice of placement method.

The ~~Grout placement~~ requirements of this Article do not apply~~are not applicable~~ to prestressing grout.

3.5 A. *Placing time* — Grout placement is often limited to1½ 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.

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 corresponds with the least clear dimension of the grout space. The minimum width of grout space is used when the grout is placed between wythes. The minimum cell dimensions are used when grouting cells of hollow masonry units. As the height of the pour increases, the minimum grout space increases. The grout space dimensions are clear dimensions. See the Commentary for Section ~~1.16~~1.19.1 of the Code for additional information.

Grout pour heights and minimum dimensions that meet the requirements of Table 7 do not automatically mean that the grout space will be filled. ~~The top of a grout pour should not be located at the top of a unit, but within 1½ in. (38 mm) of the bed joint.~~

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 elements 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.

3.5 D. *Grout lift height* — A lift is the height to which grout is placed into masonry in one continuous operation. 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. If a lift of grout is permitted to set prior to placing the subsequent lift, then that height is the pour height. This setting normally occurs if the grouting is stopped for more than one hour.

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 ~~occurrence~~from occurring. Higher lifts may be used with self-consolidating grout ~~due to~~because its fluidity and its lower initial water~~- to~~-cement ratio ~~that~~ result~~s~~ in reduced ~~less~~ potential for ~~problems due to~~ fluid pressure problems.

The 4-hour time period is stipulated for grout lifts over 5 ft (1.52 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. Cleanouts are required for pours over 5 ft (1.52 m). If intermediate, reinforced bond beams are present in the wall to be grouted, the lift height would be limited to the spacing between the bond beams or 5 ft (1.52 m), whichever is greater.

3.5 E. *Consolidation* — ~~Consolidation of grout other than s~~Except for self-consolidating grout, consolidation 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.

3.5 F. *Grout key* — The top of a grout pour should not be located at the top of a unit, but at ~~within~~ a minimum of 1½ in. (38 mm) below ~~of~~ the bed joint.

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[3.6] [5] 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.

The design method in Code Chapter 4 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 since the heat may lower the tendon strength and cause failure of the stressed tendon.

3.7 — Field quality control

3.7 A. The specified frequency of testing ~~given has long been an industry standard~~must equal or exceed the minimum requirements of the quality assurance tables.

3.7 B. ASTM C 1019 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.

References

3.1. ACI Committee 117, "Standard Specifications for Tolerances for Concrete Construction and Materials (ACI 117-90)," American Concrete Institute, Detroit, MI, 1981, 10 pp.

3.2. Uniform Building Code, International Conference of Building Officials, Whittier, CA, 1985.

3.3 Council for Masonry Wall Bracing, *Standard Practice for Bracing Masonry Walls Under Construction*, Mason Contractors Association of America, 2001, 52 pgs.

3.4~~3~~. Stecich, J.P, Hanson, John M. and Rice, Paul F., "Bending and Straightening" Concrete International, August 1984.

3.5~~4~~. "Grouting Concrete Masonry Walls", NCMA TEK 3-2A, National Concrete Masonry Association, Herndon, VA, 2005, 6 pp.

3.~~3~~6~~5~~. *Field Procedures Manual for Unbonded Single Strand Tendons, 2nd Edition*, Post-Tensioning Institute, Phoenix, AZ, 1994, 62 pp.

Foreword to Specification Checklists

F2. Building codes (of which this standard 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 sections, parts, and articles of the Specification, and the second column contains notes to the Architect/Engineer to indicate the type of action required by the Architect/Engineer.