

Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/TMS 402-02)

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SYNOPSIS

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

Among the subjects covered are: definitions; contract documents; quality assurance; materials; placement of embedded items; analysis and design; strength and serviceability; flexural and axial loads; shear; details and development of reinforcement; walls; columns; pilasters; beams and lintels; seismic design requirements; glass unit masonry; and veneers. An empirical design method and a prescriptive method applicable to buildings meeting specific location and construction criteria are also included.

The quality, inspection, testing, and placement of materials used in construction are covered by reference to ACI 530.1/ASCE 6/TMS 602 Specification and other standards.

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

¹ Regular members fully participate in Committee activities, including responding to correspondence and voting.

² Associate members monitor Committee activities, but do not have voting privileges.

Adopted as a standard of the American Concrete Institute (February 11, 2002), the Structural Engineering Institute of the American Society of Civil Engineers September 28, 2001, and The Masonry Society (February 15, 2002) to supersede the 1999 edition in accordance with each organization's standardization procedures. The standard was originally adopted by the American Concrete Institute in

November, 1988, the American Society of Civil Engineers in August, 1989, and The Masonry Society in July, 1992.

SI equivalents shown in this document are calculated conversions. Equations are based on U.S. Customary (inch-pound) Units; SI equivalents for equations are listed at the end of the Code.

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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 design and construction of masonry structural 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: 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.

1.1.4 *SI equivalents*

SI values shown in parentheses are not part of this Code.

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 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 1.14, or shall include an itemized quality assurance program that exceeds the requirements of Section 1.14.

1.2.5 Calculations pertinent to design shall be filed with the drawings when required by the building official. When automatic data processing is used, design assumptions, program documentation and identified input and output data may be submitted in lieu of calculations.

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, to require tests, and to 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, the American Society for Testing and Materials, and the American Welding Society referred to 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 530.1-02/ ASCE 6-02/ TMS 602-02 — Specification for Masonry Structures
- ASCE 7-93 — Minimum Design Loads for Buildings and Other Structures
- ASCE 7-98 — Minimum Design Loads for Buildings and Other Structures
- ASTM A 416/A 416M-00 — Specification for Steel Strand, Uncoated Seven-Wire Stress-Relieved for Prestressed Concrete
- ASTM A 421/A 421M-98a — Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete
- ASTM A 722/A 722M-98 — Specification for Uncoated High-Strength Steel Bar for Prestressed Concrete

ASTM C 426-99 — Test Method for Drying Shrinkage of Concrete Block
 ASTM C 476-99 — Specification for Grout for Masonry
 ASTM C 482-81(1996) — Test Method for Bond Strength of Ceramic Tile to Portland Cement
 ASTM E 111-82 (1996) — Test Method for Young's Modulus, Tangent Modulus, and Chord Modulus
 ASTM E 488-96 — Test Methods for Strength of Anchors in Concrete and Masonry Elements
 AWS D 1.4-98 Structural Welding Code — Reinforcing Steel

1.5 — Notation

A_b	=	cross-sectional area of an anchor bolt, in. ² (mm ²)	b_{vf}	=	factored shear force in an anchor bolt, lb (N)
A_g	=	gross cross-sectional area of masonry, in. ² (mm ²)	b_w	=	width of wall beam, in. (mm)
A_n	=	net cross-sectional area of masonry, in. ² (mm ²)	C_d	=	deflection amplification factor
A_p	=	projected area on the masonry surface of a right circular cone for anchor bolt allowable shear and tension calculations, in. ² (mm ²)	c	=	distance from the fiber of maximum compressive strain to the neutral axis, in. (mm)
A_{ps}	=	area of prestressing steel, in. ² (mm ²)	D	=	dead load or related internal moments and forces
A_{pt}	=	projected area on masonry surface of a right circular cone for calculating tensile breakout capacity of anchor bolts, in. ² (mm ²)	d	=	distance from extreme compression fiber to centroid of tension reinforcement, in. (mm)
A_{pv}	=	projected area on masonry surface of one-half of a right circular cone for calculating shear breakout capacity of anchor bolts, in. ² (mm ²)	d_b	=	nominal diameter of reinforcement or anchor bolt, in. (mm)
A_s	=	effective cross-sectional area of reinforcement, in. ² (mm ²)	d_v	=	actual depth of masonry in direction of shear considered, in. (mm)
A_v	=	cross-sectional area of shear reinforcement, in. ² (mm ²)	E	=	load effects of earthquake or related internal moments and forces
A_1	=	bearing area, in. ² (mm ²)	E_m	=	modulus of elasticity of masonry in compression, psi (MPa)
A_2	=	effective bearing area, in. ² (mm ²)	E_s	=	modulus of elasticity of steel, psi (MPa)
A_{st}	=	total area of laterally tied longitudinal reinforcing steel in a reinforced masonry column or pilaster, in. ² (mm ²)	E_v	=	modulus of rigidity (shear modulus) of masonry, psi (MPa)
a	=	depth of an equivalent compression zone at nominal strength, in. (mm)	e	=	eccentricity of axial load, in. (mm)
B_a	=	allowable axial force on an anchor bolt, lb (N)	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)
B_{an}	=	nominal axial strength of an anchor bolt, lb (N)	e_u	=	eccentricity of P_{uf} , in. (mm)
B_v	=	allowable shear force on an anchor bolt, lb (N)	F	=	lateral pressure of liquids or related internal moments and forces
B_{vn}	=	nominal shear strength of an anchor bolt, lb (N)	F_a	=	allowable compressive stress due to axial load only, psi (MPa)
b	=	width of section, in. (mm)	F_b	=	allowable compressive stress due to flexure only, psi (MPa)
b_a	=	total applied design axial force on an anchor bolt, lb (N)	F_s	=	allowable tensile or compressive stress in reinforcement, psi (MPa)
b_{af}	=	factored axial force in an anchor bolt, lb (N)	F_v	=	allowable shear stress in masonry, psi (MPa)
b_v	=	total applied design shear force on an anchor bolt, lb (N)	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'_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_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)	N_v	= compressive force acting normal to shear surface, lb (N)
f_v	= calculated shear stress in masonry, psi (MPa)	P	= axial load, lb (N)
f_y	= specified yield strength of steel for reinforcement and anchors, psi (MPa)	P_a	= allowable compressive force in reinforced masonry due to axial load, lb (N)
H	= lateral pressure of soil or related internal moments and forces	P_e	= Euler buckling load, lb (N)
h	= effective height of column, wall, or pilaster, in. (mm)	P_n	= nominal axial strength, lb (N)
I_{cr}	= moment of inertia of cracked cross-sectional area of a member, in. ⁴ (mm ⁴)	P_{ps}	= prestressing tendon force at time and location relevant for design, lb (N)
I_{eff}	= effective moment of inertia, in. ⁴ (mm ⁴)	P_u	= factored axial load, lb (N)
I_g	= moment of inertia of gross cross-sectional area of a member, in. ⁴ (mm ⁴)	P_{uf}	= factored load from tributary floor or roof areas, lb (N)
I_n	= moment of inertia of net cross-sectional area of a member, in. ⁴ (mm ⁴)	P_{uw}	= factored weight of wall area tributary to wall section under consideration, lb (N)
j	= ratio of distance between centroid of flexural compressive forces and centroid of tensile forces to depth, d	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, in. ³ (mm ³)
K	= the lesser of the masonry cover, clear spacing between adjacent reinforcement, or 5 times d_b , in. (mm)	R	= seismic response modification factor
k_c	= coefficient of creep of masonry, per psi (MPa)	r	= radius of gyration, in. (mm)
k_e	= coefficient of irreversible moisture expansion of clay masonry	S_n	= section modulus of the net cross-sectional area of a member, in. ³ (mm ³)
k_m	= coefficient of shrinkage of concrete masonry	s	= spacing of reinforcement, in. (mm)
k_t	= coefficient of thermal expansion of masonry per degree Fahrenheit (degree Celsius)	s_l	= total linear drying shrinkage of concrete masonry units determined in accordance with ASTM C 426
L	= live load or related internal moments and forces	T	= forces and moments caused by restraint of temperature, shrinkage, and creep strains or differential movements
l	= clear span between supports, in. (mm)	t	= nominal thickness of member, in. (mm)
l_b	= effective embedment length of plate, headed or bent anchor bolts, in. (mm)	v	= shear stress, psi (MPa)
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)	V	= shear force, lb (N)
l_d	= embedment length or lap length of straight reinforcement, in. (mm)	V_m	= shear strength provided by masonry, lb (N)
l_{de}	= basic development length of reinforcement, in. (mm)	V_n	= nominal shear strength, lb (N)
l_e	= equivalent embedment length provided by standard hooks, in. (mm)	V_s	= shear strength provided by shear reinforcement, lb (N)
l_p	= clear span of the prestressed member in the direction of the prestressing tendon, in. (mm)	V_u	= factored shear, lb (N)
M	= maximum moment at the section under consideration, in.-lb (N-mm)	W	= wind load or related internal moments and forces
M_a	= maximum moment in member due to the applied loading for which deflection is computed, in.-lb (N-mm)	w_u	= out-of-plane factored uniformly distributed load, lb/in. (N/mm)
M_{cr}	= nominal cracking moment strength, in.-lb (N-mm)	β	= 0.25 for fully grouted masonry or 0.15 for other than fully grouted masonry
M_n	= nominal moment strength, in.-lb (N-mm)	β_b	= ratio of area of reinforcement cut off to total area of tension reinforcement at a section
M_{ser}	= service moment at midheight of a member, including P-delta effects, in.-lb (N-mm)	γ	= reinforcement size factor
M_u	= factored moment, in.-lb (N-mm)	Δ	= calculated story drift, in. (mm)
		Δ_a	= allowable story drift, in. (mm)
		δ_s	= horizontal deflection at midheight under service loads, in. (mm)
		δ_u	= deflection due to factored loads, in. (mm)
		ϵ_{mu}	= maximum usable compressive strain of masonry
		ϕ	= strength reduction factor
		ρ	= reinforcement ratio

1.6 — Definitions

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

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

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.

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

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 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.

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 7-98.

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 — A nominal dimension is equal to a 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 — Nonload-bearing masonry composed of glass units bonded by mortar.

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.

Plain (unreinforced) masonry — Masonry in which the tensile resistance of the masonry is taken into consideration and the effects of stresses in reinforcement, if present, are neglected.

Post-tensioning — Method of prestressing in which 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 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) masonry — A masonry shear wall designed to resist lateral forces while neglecting stresses in reinforcement although provided with minimum reinforcement and connections.

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

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

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.

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.

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 pre-tensioning, 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.

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 masonry — Masonry in which the tensile resistance of masonry is taken into consideration and the resistance of the reinforcing steel 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.

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.

1.7.2 Load provisions

Service 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 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

$$E_s = 29,000,000 \text{ psi (199 955 MPa)}$$

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:

$$E_m = 700 f'_m \text{ for clay masonry;}$$

$$E_m = 900 f'_m \text{ for concrete masonry;}$$

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.

$$\mathbf{1.8.2.2.2} \quad E_v = 0.4E_m$$

1.8.2.3 Grout — Modulus of elasticity of grout shall be determined by the expression $500 f'_g$.

1.8.3 Thermal expansion coefficients

1.8.3.1 Clay masonry

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

1.8.3.2 Concrete masonry

$$k_t = 4.5 \times 10^{-6} \text{ in./in./}^\circ\text{F} \\ (8.1 \times 10^{-6} \text{ mm/mm/}^\circ\text{C)}$$

1.8.4 Moisture expansion coefficient of clay masonry

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

1.8.5 Shrinkage coefficients of concrete masonry

1.8.5.1 Masonry made of moisture-controlled concrete masonry units:

$$k_m = 0.15 s_l$$

where s_l is not more than 6.5×10^{-4} in./in. (6.5×10^{-4} mm/mm)

1.8.5.2 Masonry made of non-moisture-controlled concrete masonry units:

$$k_m = 0.5 s_l$$

1.8.6 Creep coefficients

1.8.6.1 Clay masonry

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

1.8.6.2 Concrete masonry

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

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 Member design 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.

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. Actual stresses shall be used to verify compliance with allowable stress requirements.

1.9.2 Stiffness

Determination 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 of 6 times the flange thickness or the actual flange on either side of the web wall.

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 2.3.5.

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

(a) 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: $\frac{1}{4}$ in. x $1\frac{1}{2}$ 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: 4 ft (1.22 m).

(c) Intersecting bond beams shall be provided in intersecting walls at a maximum spacing of 4 ft (1.22 m) on centers. Bond beams shall be reinforced and the area of reinforcement shall not be less than 0.1 in.^2 per ft ($211 \text{ mm}^2/\text{m}$) of wall. Reinforcement shall be developed on each side of the intersection.

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.

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 the gross vertical cross-sectional area of the wall using specified dimensions. Horizontal reinforcement shall be placed in horizontal joints or in bond beams spaced not more than 48 in. (1219 mm) on center.

1.12 — Details of reinforcement**1.12.1 Embedment**

Reinforcing bars shall be embedded in grout.

1.12.2 Size of reinforcement

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

1.12.2.2 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 1.15.1.)

1.12.2.3 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.

1.12.3 Placement of reinforcement

1.12.3.1 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).

1.12.3.2 In columns and pilasters, the clear distance between vertical bars shall not be less than one and one-half times the nominal bar diameter, nor less than 1½ in. (38.1 mm).

1.12.3.3 The clear distance limitations between bars required in Sections 1.12.3.1 and 1.12.3.2 shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

1.12.3.4 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.

1.12.3.5 Reinforcement embedded in grout shall have a thickness of grout between the reinforcement and masonry units not less than ¼ in. (6.4 mm) for fine grout or ½ in. (12.7 mm) for coarse grout.

1.12.4 Protection of reinforcement

1.12.4.1 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); 1½ in. (38.1 mm) for No. 5 (M #16) bars or smaller.
- (b) Masonry not exposed to earth or weather: 1½ in. (38.1 mm).

1.12.4.2 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 ½ 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.

1.12.4.3 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.

1.12.5 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 2½ 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.

1.12.6 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 1.12.6.1.

Table 1.12.6.1 — 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

1.13 — Seismic design requirements**1.13.1 Scope**

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

1.13.2 General**1.13.2.1 Seismic design category classification**

— Masonry shall comply with the requirements of Sections 1.13.3 through 1.13.7 based on the structure's Seismic Design Category as defined in ASCE 7-98. In addition, masonry shall comply with either the requirements of Section 1.1.3 or the requirements of Section 2.1.3.3.

1.13.2.2 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 1.13.2.2.1, 1.13.2.2.2, 1.13.2.2.3, 1.13.2.2.4, or 1.13.2.2.5.

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

1.13.2.2.1 Ordinary plain (unreinforced) masonry shear walls — Design of ordinary plain (unreinforced) masonry shear walls shall comply with the requirements of Section 2.2, Section 3.3, or Chapter 4.

1.13.2.2.2 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 3.3, and shall comply with the requirements of Sections 1.13.2.2.2.1 and 1.13.2.2.2.2.

1.13.2.2.2.1 Minimum reinforcement requirements — Vertical reinforcement of at least 0.2 in.² (129 mm²) 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 10 ft (3.05 m) 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); or bond beam reinforcement shall be provided of at least 0.2 in.² (129 mm²) in cross-sectional area spaced not more than 10 ft (3.05 m). 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.

1.13.2.2.2.2 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).

1.13.2.2.3 Ordinary reinforced masonry shear walls — Design of ordinary reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.2, and shall comply with the requirements of Sections 1.13.2.2.2.1 and 1.13.2.2.2.2.

1.13.2.2.4 Intermediate reinforced masonry shear walls — Design of intermediate reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.2. Design shall also comply with the requirements of Sections 1.13.2.2.2.1 and 1.13.2.2.2.2, except that the spacing of vertical reinforcement in intermediate reinforced masonry shear walls shall not exceed 48 in. (1219 mm).

1.13.2.2.5 Special reinforced masonry shear walls — Design of special reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.2. Design shall also comply with the requirements of Sections 1.13.2.2.2.1, 1.13.2.2.2.2, 1.13.6.3, 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.

1.13.3 Seismic Design Category A

1.13.3.1 Structures in Seismic Design Category A shall comply with the requirements of Chapter 2, 3, 4, or 5.

1.13.3.2 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 the story height.

1.13.3.3 Anchorage of masonry walls — Masonry walls shall be anchored to the roof and all floors that provide lateral support for the wall. 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 the effective peak velocity-related acceleration, lb per lineal ft of wall (14 590 times, N/m).

1.13.4 Seismic Design Category B

1.13.4.1 Structures in Seismic Design Category B shall comply with the requirements of Seismic Design Category A and to the additional requirements of Section 1.13.4.

1.13.4.2 Design of elements that are part of 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.

1.13.5 Seismic Design Category C

1.13.5.1 Structures in Seismic Design Category C shall comply with the requirements of Seismic Design Category B and to the additional requirements of Section 1.13.5.

1.13.5.2 Design of elements that are not part of lateral force-resisting system

1.13.5.2.1 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.

1.13.5.2.2 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.

1.13.5.2.3 Reinforcement requirements — Masonry elements listed in Section 1.13.5.2.2 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 than 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 48 in. (1219 mm). Vertical reinforcement shall be located within 16 in. (406 mm) of the ends of masonry walls.

1.13.5.3 Design of elements that are part of the lateral force-resisting system

1.13.5.3.1 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.

1.13.5.3.2 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.

1.13.6 Seismic Design Category D

1.13.6.1 Structures in Seismic Design Category D shall comply with the requirements of Seismic Design Category C and to the additional requirements of Section 1.13.6.

1.13.6.2 Design requirements — Masonry elements, other than those covered by Section 1.13.5.2.2, shall be designed in accordance with the requirements of Sections 2.1 and 2.3, Chapter 3, or Chapter 4.

1.13.6.3 Minimum reinforcement requirements for masonry walls — Masonry walls other than those covered by Section 1.13.5.2.3 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 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 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).

1.13.6.4 Masonry Shear Walls — Masonry shear walls shall comply with the requirements for special reinforced masonry shear walls.

1.13.6.5 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 $\frac{3}{8}$ in. (9.5 mm) diameter. Lateral ties shall be embedded in grout.

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

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

1.13.7 Seismic Design Categories E and F

1.13.7.1 Structures in Seismic Design Categories E and F shall comply with the requirements of Seismic Design Category D and to the additional requirements of Section 1.13.7.

1.13.7.2 Minimum reinforcement for stack bond elements that are not part of 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 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.

1.13.7.3 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 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.

1.14 — 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 7-98. The quality assurance program shall itemize the methods used to verify conformance of material composition, quality, storage, handling, preparation, and placement with the requirements of ACI 530.1/ASCE 6/TMS 602.

1.14.1 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 1.14.1.

1.14.2 The minimum quality assurance program for masonry in essential facilities and designed in accordance with Chapter 5, 6, or 7 shall comply with Table 1.14.2.

1.14.3 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 1.14.2.

1.14.4 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 1.14.3.

1.14.5 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.

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

1.14.7 Acceptance relative to strength requirements

1.14.7.1 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 .

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

Table 1.14.1 — Level 1 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

Table 1.14.2 — Level 2 Quality Assurance

MINIMUM TESTS AND SUBMITTALS	MINIMUM INSPECTION
<p>Certificates for materials used in masonry construction indicating compliance with the contract documents</p> <p>Verification of f'_m prior to construction, except where specifically exempted by this Code</p>	<p>As masonry construction begins, verify the following are in compliance:</p> <ul style="list-style-type: none"> • proportions of site-prepared mortar • construction of mortar joints • location of reinforcement, connectors, and prestressing tendons and anchorages • prestressing technique <p>Prior to grouting, verify the following are in compliance:</p> <ul style="list-style-type: none"> • 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 <p>Verify that the placement of grout and prestressing grout for bonded tendons is in compliance</p> <p>Observe preparation of grout specimens, mortar specimens, and/or prisms</p> <p>Verify compliance with the required inspection provisions of the contract documents and the approved submittals</p>

Table 1.14.3 — Level 3 Quality Assurance

MINIMUM TESTS AND SUBMITTALS	MINIMUM INSPECTION
<p>Certificates for materials used in masonry construction indicating compliance with the contract documents.</p> <p>Verification of f'_m :</p> <ul style="list-style-type: none"> • prior to construction • every 5000 sq. ft (464.5 m²) during construction <p>Verification of proportions of materials in mortar and grout as delivered to the site</p>	<p>From the beginning of masonry construction and continuously during construction of masonry, verify the following are in compliance:</p> <ul style="list-style-type: none"> • 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 <p>Observe preparation of grout specimens, mortar specimens, and/or prisms</p> <p>Verify compliance with the required inspection provisions of the contract documents and the approved submittals</p>

1.15 — Construction

1.15.1 Grouting, minimum spaces

The minimum dimensions of spaces provided for the placement of grout shall be in accordance with Table 1.15.1. Higher grout pours, higher grout lifts, smaller cavity widths, or smaller cell sizes than those shown in Table 1.15.1 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.

1.15.2 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.

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

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

1.15.2.3 Conduits, pipes, and sleeves in masonry shall be no closer than 3 diameters on center.

1.15.2.4 Maximum area of vertical conduits, pipes, or sleeves placed in masonry columns or pilasters shall not displace more than 2 percent of the net cross section.

1.15.2.5 Pipes shall not be embedded in masonry when:

- (a) Containing liquid, gas, or vapors at temperature higher than 150° F (66° C).
- (b) Under pressure in excess of 55 psi (379 kPa).
- (c) Containing water or other liquids subject to freezing.

Table 1.15.1 — 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)	$\frac{3}{4}$ (19.1)	$1\frac{1}{2} \times 2$ (38.1 x 50.8)
Fine	5 (1.52)	2 (50.8)	2 x 3 (50.8 x 76.2)
Fine	12 (3.66)	$2\frac{1}{2}$ (63.5)	$2\frac{1}{2} \times 3$ (63.5 x 76.2)
Fine	24 (7.32)	3 (76.2)	3 x 3 (76.2 x 76.2)
Coarse	1 (0.30)	$1\frac{1}{2}$ (38.1)	$1\frac{1}{2} \times 3$ (38.1 x 76.2)
Coarse	5 (1.52)	2 (50.8)	$2\frac{1}{2} \times 3$ (63.5 x 76.2)
Coarse	12 (3.66)	$2\frac{1}{2}$ (63.5)	3 x 3 (76.2 x 76.2)
Coarse	24 (7.32)	3 (76.2)	3 x 4 (76.2 x 102)

¹ Fine and coarse grouts are defined in ASTM 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.

CHAPTER 2 ALLOWABLE STRESS DESIGN

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) $D + L + (W \text{ or } E)$
- (d) $D + W$
- (e) $0.9 D + E$
- (f) $D + L + (H \text{ or } F)$
- (g) $D + (H \text{ or } F)$
- (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 The allowable stresses and allowable loads in Chapters 2 and 4 are permitted to be increased by one-third when considering load combination (c), (d), or (e) of Section 2.1.2.1.

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 *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.

2.1.3.3.1 *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 7-98.

For prestressed masonry, the response modification factor (R) and the deflection amplification factor (C_d), indicated in ASCE 7-98 Table 9.5.2.2 for ordinary plain (unreinforced) masonry shear walls, shall be used in determining the base shear and design story drift.

2.1.3.3.2 *Nominal strength* — The nominal strength of masonry shall be taken as $2^{1/2}$ times the allowable stress value, except as allowed in Section 4.5.3.3 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.

2.1.3.3.3 *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 solidly grouted in masonry

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).

$$B_a = 0.5A_p \sqrt{f'_m} \tag{2-1}$$

$$B_a = 0.2A_b f_y \tag{2-2}$$

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).

$$A_p = \pi l_b^2 \quad (2-3)$$

$$A_p = \pi l_{be}^2 \quad (2-4)$$

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).

$$B_v = 350 \sqrt{f'_m A_b} \quad (2-5)$$

$$B_v = 0.12 A_b f_y \quad (2-6)$$

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).

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

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:

- crossed by connecting headers, or
- 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:

- mortared collar joints, 5 psi (34.5 kPa).
- grouted collar joints, 10 psi (69.0 kPa).

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

2.1.5.2.3 Headers of wythes bonded to headers shall meet the requirements of Section 2.1.5.2 and shall be provided as follows:

- Headers shall be uniformly distributed and the sum of their cross-sectional areas shall be at least percent of the wall surface area.
- 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 to headers shall meet the requirements of Section 2.1.5.2 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 joist reinforcement are permitted to be used in lieu 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:

- Collar joints shall not contain headers, grout, or mortar.
- Gravity loads from supported horizontal members shall be resisted by the wythe nearest to the center 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 relative stiffness.
- 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.
- Loads acting transverse to the plane of a wall shall be resisted by all wythes in proportion to their relative flexural stiffnesses.
- Specified distances between wythes shall not exceed a width of 4.5 in. (114 mm) unless a detailed wall 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 $1\frac{1}{4}$ in. (31.8 mm).
- (d) Maximum clearance between connecting parts of the tie shall be $\frac{1}{16}$ 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 this section.

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

2.1.6.2 The ratio between the effective height and least nominal dimension shall not exceed 25.

2.1.6.3 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 each side dimension. Consider each axis independently.

2.1.6.4 Vertical column reinforcement shall not be less than $0.0025A_n$, nor exceed $0.04A_n$. 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 $\frac{1}{4}$ 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 such 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 as provided herein to 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 may 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.

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. For columns, a force of not less than 1,000 lb (4448 N) shall be used.

2.1.9 Concentrated loads

2.1.9.1 For computing compressive stress f_u 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 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 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 embedment length, hook or mechanical device, or a combination thereof. Hooks shall not be used to develop bars in compression.

2.1.10.2 Embedment of bars and wires in tension — The embedment length of bars and wire shall be determined by Eq. (2-8), but shall not be less than 12 in. (305 mm) for bars and 6 in. (152 mm) for wire.

$$l_d = 0.0015d_b F_s \quad (2-8)$$

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

2.1.10.3 Embedment of flexural reinforcement

2.1.10.3.1 General

2.1.10.3.1.1 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.

2.1.10.3.1.2 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.

2.1.10.3.1.3 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.

2.1.10.3.1.4 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.

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

- Shear at the cutoff point does not exceed two-thirds of the allowable shear at the section considered.
- 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 $d / (8 \beta_b)$.
- 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.

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

2.1.10.3.2 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.

2.1.10.3.3 Development of negative moment reinforcement

2.1.10.3.3.1 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.

2.1.10.3.3.2 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.

2.1.10.4 Hooks

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

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

2.1.10.5 Development of shear reinforcement

2.1.10.5.1 Bar and wire reinforcement

2.1.10.5.1.1 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.

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

- 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).
- 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).

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

2.1.10.5.1.4 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 $d/2$.

2.1.10.5.1.5 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 may be considered adequate if legs extend the full available depth of grout.

2.1.10.5.2 *Welded wire fabric*

2.1.10.5.2.1 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 $8d_b$.

2.1.10.5.2.2 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.

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

2.1.10.6.1 *Lap splices*

2.1.10.6.1.1 The minimum length of lap for bars in tension or compression shall be determined by Eq. (2-9), but not less than 12 in. (305 mm).

$$l_d = 0.002 d_b F_s \quad (2-9)$$

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

2.1.10.6.1.2 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).

2.1.10.6.2 *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.

2.1.10.6.3 *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.

2.1.10.6.4 *End-bearing splices*

2.1.10.6.4.1 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.

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

2.1.10.6.4.3 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) and Eq. (2-11).

$$\frac{f_a + f_b}{F_a F_b} \leq 1 \quad (2-10)$$

$$P \leq \left(\frac{1}{4}\right) P_e \quad (2-11)$$

where:

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

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

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

$$F_a = \left(\frac{1}{4}\right) f'_m \left(\frac{70r}{h} \right)^2 \quad (2-13)$$

(c) $F_b = \left(\frac{1}{3}\right) f'_m \quad (2-14)$

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

2.2.3.2 Allowable tensile stresses due to flexure transverse to the plane of masonry member shall be in accordance with the values in Table 2.2.3.2.

2.2.4 Axial tension

The tensile strength of 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. (2-16).

$$f_v = \frac{VQ}{I_n b} \quad (2-16)$$

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) $v + 0.45 N_v/A_n$

where v :

- = 37 psi (255 kPa) for masonry running bond that is not grouted solid or
- = 37 psi (255 kPa) for masonry in other than running bond with open end that are grouted solid, or
- = 60 psi (414 kPa) for masonry running bond that is grouted solid
- (d) 15 psi (103 kPa) for masonry in other than running bond with other than open end units that are grouted solid.

Table 2.2.3.2 — Allowable flexural tension 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)

¹ For partially grouted masonry, allowable stresses shall be determined on the basis of linear interpolation between hollow units that are fully grouted and ungrouted hollow units based on amount 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.

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.

2.3.3.2 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) or Eq. (2-18):

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

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

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

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

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 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 Beams

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

2.3.3.4.5 Beams shall be designed to meet the deflection requirements of Section 1.10.1.

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:

$$f_v = \frac{V}{bd} \quad (2-19)$$

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

$$F_v = \sqrt{f'_m} \quad (2-20)$$

but shall not exceed 50 psi (345 kPa).

(b) for shear walls,

where, $M/Vd < 1$,

$$F_v = (\frac{1}{3})[4 - (M / Vd)]\sqrt{f'_m} \quad (2-21)$$

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

where, $M/Vd \geq 1$,

$$F_v = \sqrt{f'_m} \quad (2-22)$$

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 calculated shear, f_v shall not exceed F_v , where:

(a) for flexural members:

$$F_v = 3.0 \sqrt{f'_m} \quad (2-23)$$

but shall not exceed 150 psi (1034 kPa).

(b) for shear walls:

where, $M/Vd < 1$,

$$F_v = (\frac{1}{2})[4 - (M / Vd)]\sqrt{f'_m} \quad (2-24)$$

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

where $M/Vd \geq 1$,

$$F_v = 1.5 \sqrt{f'_m} \quad (2-25)$$

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:

$$A_v = \frac{V_s}{F_s d} \quad (2-26)$$

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 load combinations, structures and members shall be designed to resist the combination of loads specified in ASCE 7-98. 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.

3.1.3 Design strength

Masonry members shall be proportioned such 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 the nominal flexural strength, M_n , of the member, except that the nominal shear strength, V_n , need not exceed 2.5 times 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, 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 Development and splices of reinforcement — For development and splicing of reinforcement, ϕ shall be taken as 0.80.

3.1.4.6 Bearing — For cases where 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 7-98.

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 7-98.

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 Deflection of reinforced masonry — Deflection calculations for reinforced masonry members shall be based on cracked section properties. 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 Headed and bent-bar anchor bolts

All embedded 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.

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

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

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).

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

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.

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

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

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

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).

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

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 for bent-bar anchor bolts — The effective embedment length 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.

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

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

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

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

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).

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

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

3.1.7 Material properties

3.1.7.1 Compressive strength

3.1.7.1.1 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.

3.1.7.1.2 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'_m , shall not exceed 6,000 psi (41.37 MPa).

3.1.7.2 Masonry modulus of rupture

3.1.7.2.1 Out-of-plane bending — The modulus of rupture, f_r , for masonry elements subjected to out-of-plane bending shall be taken from Table 3.1.7.2.1.

3.1.7.2.2 In-plane bending — For masonry subjected to in-plane loads, the modulus of rupture, f_r , normal to the bed joints shall be taken as 250 psi (1.72 MPa). The modulus of rupture used for masonry parallel

to the bed joints shall be taken as 250 psi (1.72 MPa). For grouted stack bond masonry, tension parallel to the bed joints shall be assumed to be resisted only by the continuous horizontal grout section.

3.1.7.3 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 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.

Table 3.1.7.2.1 — Modulus of rupture (f_r), psi (kPa) *for out of plane bending*

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	170 (1172)	145 (999)	103 (710)	73 (503)
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	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 hollow units that are fully grouted and ungrouted based on amount (percentage) of grouting.

3.2 — Reinforced masonry

3.2.1 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.

3.2.2 Design assumptions

The following assumptions apply to the design of reinforced masonry:

- There is strain continuity between the reinforcement, grout, and masonry such that all applicable loads are resisted in a composite manner.
- The nominal strength of reinforced masonry cross-sections for combined flexure and axial load shall be based on applicable conditions of equilibrium.
- 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.
- Strain in reinforcement and masonry shall be assumed to be directly proportional to the distance from the neutral axis.
- Reinforcement stress below specified yield strength, f_y , shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y , stress in reinforcement shall be taken equal to f_y .
- The tensile strength of masonry shall be neglected in calculating flexural strength but shall be considered in calculating deflection.
- 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 bounded by edges of the cross section and a straight line located parallel to the neutral axis 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 axis.

3.2.3 Reinforcement requirements and details

3.2.3.1 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 it 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.

3.2.3.2 Standard hooks — The equivalent embedment length to develop standard hooks in tension, l_e , shall be determined by Eq. (3-12):

$$l_e = 13d_b \quad (3-12)$$

3.2.3.3 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. (3-13), but shall not be less than 12 in. (305 mm).

$$l_d = \frac{l_{dc}}{\phi} \quad (3-13)$$

Where:

$$l_{dc} = \frac{0.13 d_b^2 f_y \gamma}{K \sqrt{f'_m}} \quad (3-14)$$

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

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

$\gamma = 1.4$ 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.

3.2.3.3.1 Development of shear reinforcement — Shear reinforcement shall extend the depth of the member less cover distances.

3.2.3.3.1.1 Except at wall intersections, the end of a horizontal reinforcing bar needed to satisfy shear strength requirements of Section 3.2.4.1.2 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.

3.2.3.3.1.2 At wall intersections, horizontal reinforcing bars needed to satisfy shear strength requirements of Section 3.2.4.1.2 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.

3.2.3.4 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 length determined by Eq. (3-15), whichever is greater.

$$l_d = \frac{l_{de}}{\phi} \quad (3-15)$$

- (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.

3.2.3.5 Maximum reinforcement percentages

3.2.3.5.1 For structures designed using an R value greater than 1.5, the ratio of reinforcement, ρ , shall not exceed the lesser of the values required to satisfy the following two critical strain conditions:

- (a) For walls subjected to in-plane forces, for columns, and for beams, a strain of 5 times yield in the extreme tension reinforcement and a maximum masonry strain defined by Section 3.2.2(c).
- (b) For walls subjected to out-of-plane forces, a strain of 1.3 times yield in the extreme tension reinforcement and a maximum masonry strain defined by Section 3.2.2(c).

In calculating the maximum reinforcement ratio for each case, equilibrium shall include unfactored gravity axial loads. The stress in the tension reinforcement shall be assumed to be $1.25 f_y$. Tension in the masonry shall be neglected. The strength of the compression zone shall be calculated as 80 percent of f'_m times 80 percent of the area of the compressive zone. Stress in reinforcement in the compression zone shall be based on a linear strain distribution.

3.2.3.5.2 For structures designed using an R value less than or equal to 1.5, the ratio of reinforcement, ρ , shall not exceed the ratio as calculated using the following critical strain condition:

A strain of 2 times yield in the extreme tension reinforcement and a maximum masonry strain defined by Section 3.2.2(c). In calculating the maximum reinforcement ratio, equilibrium shall include unfactored gravity axial loads. The stress in the tension reinforcement shall be assumed to be $1.25 f_y$. Tension in the masonry shall be neglected. The strength of the compression zone shall be calculated as 80 percent of f'_m times 80 percent of the area of the compressive zone. Stress in reinforcement in the compression zone shall be based on a linear strain distribution.

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

3.2.4 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.

3.2.4.1 Nominal strength

3.2.4.1.1 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 3.2.2 and the provisions of Section 3.2.4.1. Using the slenderness-dependent modification factors of Eq. (3-16) $(1-(h/140r)^2)$ and Eq. (3-17) $(70r/h)^2$, 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) or Eq. (3-17), as appropriate.

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

$$P_n = 0.80 \left[0.80 f'_m (A_n - A_s) + f_y A_s \right] \left[1 - \left(\frac{h}{140r} \right)^2 \right] \quad (3-16)$$

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

$$P_n = 0.80 \left[0.80 f'_m (A_n - A_s) + f_y A_s \right] \left(\frac{70r}{h} \right)^2 \quad (3-17)$$

3.2.4.1.2 Nominal shear strength — Nominal shear strength, V_n , shall be computed using Eq. (3-18) and either Eq. (3-19) or Eq. (3-20), as appropriate.

$$V_n = V_m + V_s \quad (3-18)$$

where V_n shall not exceed the following:

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

$$V_n \leq 6 A_n \sqrt{f'_m} \quad (3-19)$$

- (b) Where $M/Vd_v \geq 1.00$

$$V_n \leq 4 A_n \sqrt{f'_m} \quad (3-20)$$

- (c) The maximum value of V_n for M/Vd_v between 0.25 and 1.0 may be interpolated.

3.2.4.1.2.1 Nominal masonry shear strength — Shear strength provided by the masonry, V_m , shall be computed using Eq. (3-21):

$$V_m = \left[4.0 - 1.75 \left(\frac{M}{Vd_v} \right) \right] A_n \sqrt{f'_m} + 0.25P \quad (3-21)$$

M/Vd_v need not be taken greater than 1.0.

(Total)

(Masonry only)
 V_m

3.2.4.1.2.2 Nominal shear strength provided by reinforcement — Nominal shear strength provided by reinforcement, V_s , shall be computed as follows:

$$V_s = 0.5 \left(\frac{A_v}{s} \right) f_y d_v \quad (3-22)$$

3.2.4.2 Beams

3.2.4.2.1 Members designed primarily to resist flexure shall comply with the requirements of Section 3.2.4.2. The factored axial compressive force on a beam shall not exceed $0.05 A_n f'_m$.

3.2.4.2.2 Longitudinal reinforcement

3.2.4.2.2.1 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.

3.2.4.2.2.2 The nominal flexural strength of a beam shall not be less than 1.3 times 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 3.1.7.2.

3.2.4.2.3 Transverse reinforcement

Transverse reinforcement shall be provided where V_u exceeds ϕV_m . The factored shear, V_u , shall include the effects of lateral load. When transverse reinforcement is required, the following provisions shall apply:

- Transverse reinforcement shall be a single bar with a 180-degree hook at each end.
- Transverse reinforcement shall be hooked around the longitudinal reinforcement.
- The minimum area of transverse reinforcement shall be $0.0007bd_v$.
- The first transverse bar shall not be located more than one fourth of the beam depth, d_v , from the end of the beam.
- The maximum spacing shall not exceed one half the depth of the beam nor 48 in. (1219 mm).

3.2.4.2.4 Construction — Beams shall be grouted solid.

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

- 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.
- The nominal depth of a beam shall not be less than 8 in. (203 mm).

3.2.4.3 Piers

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

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

- One bar shall be provided in the end cells.
- The minimum area of longitudinal reinforcement shall be $0.0007bd$.
- Longitudinal reinforcement shall be uniformly distributed throughout the depth of the element.

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

- The nominal thickness of a pier shall not be less than 6 in. (152 mm) and shall not exceed 16 in. (406 mm).
 - The distance between lateral supports of a pier shall not exceed 25 times the nominal thickness of a pier, except as provided for in Section 3.2.4.3.3(c).
 - When the distance between lateral supports of a pier exceeds 25 times the nominal thickness of the pier, the design shall be based on the provisions of Section 3.2.5.
 - The nominal length of a pier shall not be less than three times its nominal thickness nor greater than five times its nominal thickness. The clear height of a pier shall not exceed five times its nominal length.
- Exception:* When the factored axial force at location of maximum moment is less than $0.05 A_n$, the length of a pier may be equal to the thickness of the pier.

3.2.4.4 Columns

3.2.4.4.1 Longitudinal reinforcement

Longitudinal reinforcement shall be a minimum of four bars, one in each corner of the column, and shall comply with the following:

- Maximum reinforcement area shall be determined in accordance with Section 3.2.3.5, but shall not exceed $0.04 A_n$.
- Minimum reinforcement area shall be $0.0025 A_n$.
- Longitudinal reinforcement shall be uniformly distributed throughout the depth of the element.

3.2.4.4.2 Lateral ties — Lateral ties shall be provided in accordance with Section 2.1.6.5.

3.2.4.4.3 Construction — Columns shall be solid grouted.

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

- The nominal width of a column shall not be less than 8 in. (203 mm).
- The distance between lateral supports of a column shall not exceed 30 times its nominal width.
- The nominal depth of a column shall not be less than 8 in. (203 mm) and not greater than three times its nominal width.

3.2.5 Wall design for out-of-plane loads

3.2.5.1 General — The requirements of Section 3.2.5 are for the design of walls for out-of-plane loads.

3.2.5.2 Maximum reinforcement — The maximum reinforcement ratio shall be determined by Section 3.2.3.5.

3.2.5.3 Moment and deflection calculations — All moment and deflection calculations in Section 3.2.5.4 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.

3.2.5.4 Walls with factored axial stress of $0.05 f'_m$ 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).

$$\left(\frac{P_u}{A_g} \right) \leq 0.05 f'_m \quad (3-23)$$

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).

$$M_u = \frac{w_u h^2}{8} + P_{uf} \frac{e_u}{2} + P_u \delta_u \quad (3-24)$$

Where:

$$P_u = P_{uw} + P_{uf} \quad (3-25)$$

The design strength for out-of-plane wall loading shall be in accordance with Eq. (3-26).

$$M_u \leq \phi M_n \quad (3-26)$$

Where:

$$M_n = (A_s f_y + P_u) \left(d - \frac{a}{2} \right) \quad (3-27)$$

$$a = \frac{(P_u + A_s f_y)}{0.80 f'_m b} \quad (3-28)$$

The nominal shear strength shall be determined by Section 3.2.4.1.2.

3.2.5.5 Walls with factored axial stress greater than $0.05 f'_m$ — The procedures set forth in this section shall be used for the design of masonry walls when the factored axial load stress at the location of maximum moment exceeds $0.05 f'_m$. These provisions shall not be applied to walls with factored axial load stress equal to or exceeding $0.2 f'_m$ or slenderness ratios exceeding 30. Such walls shall be designed in accordance with the provisions of Section 3.2.5.4 and shall have a minimum nominal thickness of 6 in. (152 mm).

The nominal shear strength shall be determined by Section 3.2.4.1.2.

3.2.5.6 Deflection design — The horizontal midheight deflection, δ_s , under service lateral and service axial loads (without load factors) shall be limited by the relation:

$$\delta_s \leq 0.007 h \quad (3-29)$$

P-delta effects shall be included in deflection calculation. The midheight deflection shall be computed using either Eq. (3-30) or Eq. (3-31), as applicable.

(a) Where $M_{ser} < M_{cr}$

$$\delta_s = \frac{5 M_{ser} h^2}{48 E_m I_g} \quad (3-30)$$

(b) Where $M_{cr} < M_{ser} < M_n$

$$\delta_s = \frac{5 M_{cr} h^2}{48 E_m I_g} + \frac{5 (M_{ser} - M_{cr}) h^2}{48 E_m I_{cr}} \quad (3-31)$$

The cracking moment strength of the wall shall be computed using Eq. (3-32):

$$M_{cr} = S_n f_r \quad (3-32)$$

The modulus of rupture, f_r , shall be taken from Table 3.1.7.2.1.

3.2.6 Wall design for in-plane loads

3.2.6.1 Scope — The requirements of Section 3.2.6 are for the design of walls to resist in-plane loads.

3.2.6.2 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 3.2.3.5.

3.2.6.3 Flexural and axial strength — The nominal flexural and axial strength shall be determined in accordance with Section 3.2.4.1.1.

3.2.6.4 Shear strength — The nominal shear strength shall be computed in accordance with Section 3.2.4.1.2.

3.3—Unreinforced (plain) masonry

3.3.1 Scope

The requirements of Section 3.3 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.

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

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

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

3.3.2 Flexural strength of unreinforced (plain) masonry members

The following assumptions shall apply when determining the flexural strength of unreinforced (plain) masonry members:

- Strength design of members for factored flexure and axial load shall be in accordance with principles of engineering mechanics.
- Strain in masonry shall be directly proportional to the distance from the neutral axis.
- Flexural tension in masonry shall be assumed to be directly proportional to strain.
- 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 flexural tensile strength of masonry shall be determined from Section 3.1.7.2.

3.3.3 Nominal axial strength of unreinforced (plain) masonry members

Nominal axial strength, P_n , shall be computed using Eq. (3-33) or Eq. (3-34).

- For members having an h/r ratio not greater than 99:

$$P_n = 0.80 \left(0.80 A_n f'_m \left[1 - \left(\frac{h}{140r} \right)^2 \right] \right) \quad (3-33)$$

- For members having an h/r ratio greater than 99:

$$P_n = 0.80 \left(0.80 A_n f'_m \left(\frac{70r}{h} \right)^2 \right) \quad (3-34)$$

3.3.4 Nominal shear strength — Nominal shear strength, V_n , shall be the smallest of the following:

- $3.8 A_n \sqrt{f'_m}$
- $300 A_n$
- For running bond masonry not solidly grouted;

$$56 A_n + 0.45 N_v$$
- For stack bond masonry with open end units and grouted solid;

$$56 A_n + 0.45 N_v$$
- For running bond masonry grouted solid;

$$90 A_n + 0.45 N_v$$
- For stack bond other than open end units grouted solid;

$$23 A_n$$