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

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SYNOPSIS

This commentary documents some of the considerations of the Masonry Standards Joint Committee in developing the provisions contained in “Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/TMS 402-02).” This information is provided in the commentary because this Code is written as a legal document and cannot therefore present background details or suggestions for carrying out its requirements.

Emphasis is given to the explanation of new or revised provisions that may be unfamiliar to users of this Code. References to much of the research data used to prepare this Code are cited for the user desiring to study individual items in greater detail. The subjects covered are those found in this Code. The chapter and section numbering of this Code are followed throughout.

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.

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.
INTRODUCTION

This commentary documents some of the considerations of the Masonry Standards Joint Committee (MSJC) in developing the provisions contained in Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/TMS 402-02), hereinafter called this Code. Comments on specific provisions are made under the corresponding chapter and section numbers of this Code.

The commentary is not intended to provide a detailed account of the studies and research data reviewed by the committee in formulating the provisions of this Code. However, references to some of the research data are provided for those who wish to study the background material in depth.

As the name implies, Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/TMS 402-02) is meant to be used as part of a legally adopted building code and as such must differ in form and substance from documents that provide detailed specifications, recommended practices, complete design procedures, or design aids.

This Code is intended to cover all buildings of the usual types, both large and small. This Code and this commentary cannot replace sound engineering knowledge, experience, and judgment. Requirements more stringent than the Code provisions may sometimes be desirable.

A building code states only the minimum requirements necessary to provide for public health and safety. The MSJC Building Code is based on this principle. For any structure, the owner or the structural designer may require the quality of materials and construction to be higher than the minimum requirements necessary to protect the public as stated in this Code. However, lower standards are not permitted.

This commentary directs attention to other documents that provide suggestions for carrying out the requirements and intent of this Code. However, those documents and this commentary are not intended to be a part of this Code.

This Code has no legal status unless it is adopted by government bodies having the police power to regulate building design and construction or unless incorporated into a contract. Where this Code has not been adopted, it may serve as a reference to good practice even though it has no legal status.

This Code provides a means of establishing minimum standards for acceptance of designs and construction by a legally appointed building official or designated representatives. Therefore, this Code cannot define the contract responsibility of each of the parties in usual construction unless incorporated into a contract. However, general references requiring compliance with this Code in the project specifications are improper since minimum code requirements should be incorporated in the contract documents, which should contain all requirements necessary for construction.

Masonry is one of the oldest forms of construction. In modern times, the design of masonry has been governed by standards which separate clay masonry from concrete masonry. For this Code, the committee has adopted the policy that the design methodology for all masonry should be the same. The committee adopted this policy in recognition that the design methodology developed does not always predict the actual performance of masonry as accurately as it would like and that masonry work designed in accordance with some empirical provisions performs better than would be indicated by current design procedures. These design situations are being identified by the committee and singled out for further detailed research.
CHAPTER 1
GENERAL DESIGN REQUIREMENTS FOR MASONRY

1.1 — Scope
This Code covers the structural design and construction of masonry elements and serves as a part of the legally adopted building code. Since the requirements for masonry in this Code are interrelated, this Code may need to supersede when there are conflicts on masonry design and construction with the legally adopted building code or with documents referenced by this Code. The designer must resolve the conflict for each specific case.

1.1.3 Design procedures
The design procedures in Chapter 2 are allowable stress methods in which the stresses resulting from service loads do not exceed permissible service load stresses.

Linear elastic materials following the Hooke’s Law are assumed, that is, deformations (strains) are linearly proportional to the loads (stresses). All materials are assumed to be homogeneous and isotropic, and sections that are plane before bending remain plane after bending. These assumptions are adequate within the low range of working stresses under consideration. The allowable stresses are fractions of the specified compressive strength, resulting in conservative factors of safety.

Service load is the load which is assumed by the legally adopted building code to actually occur when the structure is in service. The stresses allowed under the action of service loads are limited to values within the elastic range of the materials.

Empirical design procedures of Chapter 5 are permitted in certain instances. Members not working integrally with the structure, such as partition or panel walls, or any member not (or not permanently) absorbing or transmitting forces resulting from the behavior of the structure under loads, may be designed empirically. A masonry shear wall would be an integral structural part while some wall partitions, because of their method of construction or attachment, would not. Empirical design is permitted for buildings of limited height and low seismic exposure.

1.2 — Contract documents and calculations
1.2.1 The provisions for preparation of project drawings, project specifications, and issuance of permits are, in general, consistent with those of most legally adopted building codes and are intended as supplements thereto.

This Code is not intended to be made a part of the contract documents. The contractor should not be asked through contract documents to assume responsibility regarding design (Code) requirements, unless the construction entity is acting in a design-build capacity. A commentary on ACI 530.1/ASCE 6/TMS 602 follows the Specification.

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

Masonry does not always behave in the same manner as its structural supports or adjacent construction. The designer should consider these differential movements and the forces resulting from their restraint. The type of connection chosen should transfer only the loads planned. While some connections transfer loads perpendicular to the wall, other devices transfer loads within the plane of the wall. Details shown in Fig. 1.2.2-1 are representative examples and allow movement within the plane of the wall. While load transfer usually involves masonry attached to structural elements such as beams or columns, the connection of nonstructural elements such as door and window frames should also be investigated.

Connectors are of a variety of sizes, shapes, and uses. In order to perform properly they should be identified on the project drawings.

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

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

1.2.5 This Code accepts documented computer programs as a means of obtaining a structural analysis or design in lieu of detailed manual calculations. The extent of input and output information required will vary according to the specific requirements of individual building officials. However, when a computer program has been used by the designer, only skeleton data should normally be required. Design assumptions and program documentation are necessary. This should consist of sufficient input and output data and other information to allow the building official to perform a detailed review and make comparisons using another program or manual calculations. Input data should be identified as to member designation, applied loads, and span lengths. The related output data should include member designation and the shears, moments, and reactions at key points. Recommendations for computer submittals are detailed in “Recommended Documentation for Computer Calculation Submittals to Building Officials” reported by ACI Committee 118.1
1.3 — Approval of special systems of design or construction

New methods of design, new materials, and new uses of materials must undergo a period of development before being specifically covered in a code. Hence, valid systems or components might be excluded from use by implication if means were not available to obtain acceptance. This section permits proponents to submit data substantiating the adequacy of their system or component to a “board of examiners.” Such a board should be created and named in accordance with local laws, and should be headed by a registered engineer. All board members should be directly associated with, and competent in, the fields of structural design or construction of masonry.

For special systems considered under this section, specific tests, load factors, deflection limits, and other pertinent requirements should be set by the board of examiners, and should be consistent with the intent of the code.

1.4—Standards cited in this Code

These standards are referenced in this Code. Specific dates are listed here since changes to the standard may result in changes of properties or procedures. Two editions of ASCE 7 are referenced, since some of the provisions in this standard are still based on the earlier edition of ASCE 7. Accordingly, the architect/engineer is cautioned to read the provisions carefully to ensure that the appropriate provisions are applied.
1.5 — Notation

Notations used in this Code are summarized here. Each symbol is unique, with the notation as used in other masonry standards when possible. Figure 1.5-1 graphically shows $e_b$ for a bent-bar anchor bolt.

![Fig. 1.5-1 — Bent-bar anchor bolt](image)

1.6 — Definitions

For consistent application of this Code, terms are defined which have particular meanings in this Code. The definitions given are for use in application of this Code only and do not always correspond to ordinary usage. Glossaries of masonry terminology are available from several sources within the industry.\(^1\), \(^2\), \(^3\), \(^4\)

The permitted tolerances for units are found in the appropriate materials standards. Permitted tolerances for joints and masonry construction are found in the Specification. Nominal dimensions are usually used to identify the size of a masonry unit. The thickness or width is given first, followed by height and length. Nominal dimensions are normally given in whole numbers nearest to the specified dimensions. Specified dimensions are most often used for design calculations.

1.7 — Loading

The provisions establish design load requirements. If the service loads specified by the legally adopted building code differ from those of ASCE 7-98, the legally adopted building code governs. The Architect/Engineer may decide to use the more stringent requirements.

1.7.3 Lateral load resistance

Lateral load resistance must be provided by a braced structural system. Partitions, infill panels, and similar elements may not be a part of the lateral-force-resisting system if isolated. However, when they resist lateral forces due to their rigidity, they should be considered in analysis.

1.7.4 Other effects

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

1.7.5 Lateral load distribution

The design assumptions for masonry buildings include the use of a braced structural system. The distribution of lateral loads to the members of the resisting structural system is a function of the rigidities of the structural system and of the horizontal diaphragms. The method of connection at intersecting walls and between walls and floor and roof diaphragms determines if the wall participates in the resisting structural system. Lateral loads from wind and seismic forces are normally considered to act in the direction of the principal axes of the structure. Lateral loads may cause forces in walls both perpendicular and parallel to the direction of the load. Horizontal torsion can be developed due to eccentricity of the applied load with respect to the center of rigidity.

The analysis of lateral load distribution should be in accordance with accepted engineering procedures. The analysis should rationally consider the effects of openings in shear walls and whether the masonry above the openings allows them to act as coupled shear walls. See Fig. 1.7-1. The interaction of coupled shear walls is complex and further information may be obtained from Reference 1.5.

Computation of the stiffness of shear walls should consider shearing and flexural deformations. A guide for solid shear walls (that is, with no openings) is given in Fig. 1.7-2. For nongrouted hollow unit shear walls, the use of equivalent solid thickness of wall in computing web stiffness is acceptable.

1.8 — Material properties

1.8.1 General

Proper evaluation of the building material movement from all sources is an important element of masonry design. Brick and concrete masonry may behave quite differently under normal loading and weather conditions. The committee has extensively studied available research information in the development of these material properties. However, the Committee recognizes the need for further research on this subject. The designer is encouraged to review industry standards for further design information and movement joint locations. Material properties can be determined by appropriate tests of the materials to be used.
1.8.2 Elastic moduli

Modulus of elasticity for masonry has traditionally been taken as 1000 $f^\prime_m$ in previous masonry codes. Research has indicated, however, that lower values may be more typical. A compilation of the available research has indicated a large variation in the relationship of elastic modulus versus compressive strength of masonry. However, variation in procedures between one research investigation and another may account for much of the indicated variation. Furthermore, the type of elastic moduli being reported (that is, secant modulus, tangent modulus, chord modulus, etc.) is not always identified. The committee decided the most appropriate elastic modulus for working-stress design purposes is the slope of the stress-strain curve below a stress value of 0.33 $f^\prime_m$, the allowable flexural compressive stress. Data at the bottom of the stress strain curve may be questionable due to the seating effect of the specimen during the initial loading phase if measurements are made on the testing machine platens. The committee therefore decided that the most appropriate elastic modulus for design purposes is the chord modulus from a stress value of 5 to 33 percent of the compressive strength of masonry (see Fig. 1.8-1). The terms chord modulus and secant modulus have been used interchangeably in the past. The chord modulus, as used herein, is defined as the slope of a line intersecting the stress-strain curve at two points, neither of which is the origin of the curve.
The elastic modulus is determined as a function of masonry compressive strength using the relations developed from an extensive survey of modulus data by Wolde-Tinsae et al. and results of a test program by Colville et al. Code values for $E_m$ are higher than indicated by a best fit of data relating $E_m$ to the compressive strength of masonry. The higher Code values are based on the fact that actual compressive strength significantly exceeds the specified compressive strength of masonry, $f'_m$, particularly for clay masonry.

By using the Code values, the contribution of each wythe to composite action is better taken into account in design calculations than would be the case if the elastic modulus of all parts of a composite wall were based on one specified compressive strength of masonry.

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

### 1.8.3 Thermal expansion coefficients

Temperature changes cause material expansion and contraction. This material movement is theoretically reversible. These thermal expansion coefficients are slightly higher than mean values for the assemblage.

Thermal expansion for concrete masonry will vary with aggregate type.

### 1.8.4 Moisture expansion coefficient of clay masonry

Fired clay products expand upon contact with moisture and the material does not return to its original size upon drying. This is a long-term expansion as clay particles react with atmospheric moisture. Continued expansion has been reported for 7½ years. Moisture expansion is reversible in concrete masonry.

### 1.8.5 Shrinkage coefficients of concrete masonry

Concrete masonry is a portland cement-based material that will shrink due to moisture loss and carbonation. Moisture-controlled units must be kept dry in order to retain the lower shrinkage values. The total linear drying shrinkage is determined by ASTM C 426. The shrinkage of clay masonry is negligible.

### 1.8.6 Creep coefficients

When continuously stressed, these materials gradually deform in the direction of stress application. This movement is referred to as creep and is load and time dependent. The values given are maximum values.

### 1.8.7 Prestressing steel

The material and section properties of prestressing steels may vary with each manufacturer. Most significant for design are the prestressing tendon’s cross section, modulus of elasticity, tensile strength, and stress relaxation properties. Values for these properties for various manufacturers’ wire, strand, and bar systems are given elsewhere. The modulus of elasticity of prestressing steel is often taken equal to 28,000 ksi (193 060 MPa) for design, but can vary and should be verified by the manufacturer. Stress-strain characteristics and stress relaxation properties of prestressing steels must be determined by test, because these properties may vary between different steel forms (bar, wire, or strand) and types (mild, high strength, or stainless).

### 1.9 — Section properties

#### 1.9.1 Stress computations

Minimum net section is often difficult to establish in hollow unit masonry. The designer may choose to use the minimum thickness of the face shells of the units as the minimum net section. The minimum net section may not be the same in the vertical and horizontal directions.
For masonry of hollow units, the minimum cross-sectional area in both directions may conservatively be based on the minimum face shell thickness.1.14

Solid clay masonry units are permitted to have coring up to a maximum of 25 percent of their gross cross-sectional area. For such units, the net cross-sectional area may be taken as equal to the gross cross-sectional area, except as provided in Section 2.1.5.2.2(c) for masonry headers. Several conditions of net area are shown in Fig. 1.9-1.

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

### 1.9.2 Stiffness

Stiffness is a function of the extent of cracking. The Code equations for design in Section 2.2, however, are based on the member’s uncracked moment of inertia. Also, since the extent of tension cracking in shear walls is not known in advance, this Code allows the determination of stiffness to be based on uncracked section properties. For reinforced masonry, the stiffness calculations based on the cracked section will yield more accurate results.

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

### 1.9.3 Radius of gyration

The radius of gyration is the square root of the ratio of bending moment of inertia to cross-sectional area. Since stiffness is based on the average net cross-sectional area of the member considered, this same area should be used in the computation of radius of gyration.

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*Fig. 1.9-1 — Net cross-sectional areas*
1.9.4 Intersecting walls

Connections of webs to flanges of shear walls may be accomplished by running bond, metal connectors, or bond beams. Achieving stress transfer at a T intersection with running bond only is difficult. A running bond connection should be as shown in Fig. 1.9-2 with a “T” geometry over their intersection.

The alternate method, making use of metal strap connectors, is shown in Fig. 1.9-3. Bond beams, shown in Fig. 1.9-4, are the third means of connecting webs to flanges.

When the flanges are connected at the intersection, they are required to be included in the design. The effective width of the flange is traditional requirement. The effective flange width is shown in Fig. 1.9-5.
1.10 — Deflection

1.10.1 Deflection of beams and lintels
These deflection limits apply to beams of all materials that support unreinforced masonry. These empirical requirements limit excessive deflections that may result in damage to the supported masonry. Where supported masonry is designed in accordance with Section 2.3, it is assumed that crack width in masonry will be controlled by the reinforcement so the deflection requirements are waived.

1.10.2 Connection to structural frames
Exterior masonry walls connected to structural frames are used primarily as non-bearing curtain walls. Regardless of the structural system used for support, there are differential movements between the structure and the wall. These differential movements may occur separately or in combination and may be due to the following:
1) Temperature increase or decrease of either the structural frame or the masonry wall.
2) Moisture and freezing expansion of brick or shrinkage of concrete block walls.
3) Elastic shortening of columns from axial loads, shrinkage, or creep.
4) Deflection of supporting beams.
5) Sidesway in multiple-story buildings.
6) Foundation movement.

Since the tensile strength of masonry is low, these differential movements must be accommodated by sufficient clearance between the frame and masonry and flexible or slip-type connections.
Structural frames and bracing should not be infilled with masonry to increase resistance to in-plane lateral forces without considering the differential movements listed above.

Wood, steel, or concrete columns may be surrounded by masonry serving as a decorative element. Masonry walls may be subject to forces as a result of their interaction with other structural components. Since the masonry element is often much stiffer, the load will be carried first by the masonry. These forces, if transmitted to the surrounding masonry, should not exceed the allowable stresses of the masonry. Alternately, there should be sufficient clearance between the frame and masonry. Flexible ties should be used to allow for the deformations.

Beams or trusses supporting masonry walls are essentially embedded, and their deflections should be limited to the allowable deflections for the masonry being supported. See Section 1.10.1 for requirements.

1.11 — Stack bond masonry
The requirements separating running bond from stack bond are shown in Fig. 1.11-1. The amount of steel required in this section is an arbitrary amount to provide continuity across the head joints. This reinforcement can be used to resist load.

1.12 — Details of reinforcement
In setting the provisions of this section, the committee used the ACI 318 Code1.15 as a guide. Some of the requirements were simplified and others dropped, depending on their suitability for application to masonry.

1.12.2 Size of reinforcement
1.12.2.1 Limits on size of reinforcement are based on accepted practice and successful performance in construction. The No. 11 (M#36) limit is arbitrary, but Reference 2.50 shows that distributed small bars provide better performance than fewer large bars. Properties of reinforcement are given in Table 1.12.1.

1.12.2.2 Adequate flow of grout for the achievement of good bond is achieved with this limitation. It also limits the size of reinforcement when combined with Section 1.15.1.

1.12.2.3 The function of joint reinforcement is to control the size and spacing of cracks caused by volume changes in masonry as well as to resist tension. Joint reinforcement is commonly used in concrete masonry to minimize shrinkage cracking. The restriction on wire size ensures adequate performance. The maximum wire size of one-half the joint thickness allows free flow of mortar around joint reinforcement. Thus, a $\frac{7}{16}$ in. (4.8 mm) diameter wire can be placed in a $\frac{3}{16}$ in. (9.5 mm) joint.

1.12.3 Placement of reinforcement
Placement limits for reinforcement are based on successful construction practice over many years. The limits are intended to facilitate the flow of grout between bars. A minimum spacing between bars in a layer prevents longitudinal splitting of the masonry in the plane of the bars. Use of bundled bars in masonry construction is rarely required. Two bars per bundle is considered a practical maximum. It is important that bars be placed accurately. Reinforcing bar positioners are available to control bar position.

Fig. 1.11-1 — Running bond masonry
1.12.4 Protection of reinforcement

1.12.4.1 Reinforcing bars are traditionally not galvanized. The masonry cover retards corrosion of the steel. Cover is measured from the exterior masonry surface to the outer-most surface of the steel to which the cover requirement applies. It is measured to the outer edge of stirrups or ties, if transverse reinforcement encloses main bars. Masonry cover includes the thickness of masonry units, mortar, and grout. At bed joints, the protection for reinforcement is the total thickness of mortar and grout from the exterior of the mortar joint surface to outer-most surface of the steel.

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

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

1.12.4.3 Corrosion resistance requirements are included since masonry cover varies considerably for these items. The exception for anchor bolts is based on current industry practice.

1.12.5 Standard hooks

Standard hooks are shown in Fig. 1.12-1.

1.12.6 Minimum bend diameter for reinforcing bars

Standard bends in reinforcing bars are described in terms of the inside diameter of bend since this is easier to measure than the radius of bend.

A broad survey of bending practices, a study of ASTM bend test requirements, and a pilot study of and experience with bending Grade 60 (413.7 MPa) bars were considered in establishing the minimum diameter of bend. The primary consideration was feasibility of bending without breakage. Experience since has established that these minimum bend diameters are satisfactory for general use without detrimental crushing of grout.

<table>
<thead>
<tr>
<th>Designation</th>
<th>Diameter, in. (mm)</th>
<th>Area, in.² (mm²)</th>
<th>Perimeter, in. (mm)</th>
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<tr>
<td>Wire</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W1.1 (11 gage) (MW7)</td>
<td>0.121 (3.1)</td>
<td>0.011 (7.1)</td>
<td>0.380 (9.7)</td>
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<td>W1.7 (9 gage) (MW11)</td>
<td>0.148 (3.8)</td>
<td>0.017 (11.0)</td>
<td>0.465 (11.8)</td>
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<tr>
<td>W2.1 (8 gage) (MW13)</td>
<td>0.162 (4.1)</td>
<td>0.020 (12.9)</td>
<td>0.509 (12.9)</td>
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<td>W2.8 (3/16 wire) (MW17)</td>
<td>0.187 (4.8)</td>
<td>0.027 (17.4)</td>
<td>0.587 (14.9)</td>
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<td>W4.9 (1/4 wire) (MW32)</td>
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<td>0.049 (31.6)</td>
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<td>No. 3 (M#10)</td>
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<td>1.56 (1006)</td>
<td>4.430 (113)</td>
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</table>
1.13 — Seismic design requirements

1.13.1 Scope

The requirements in this section have been devised to improve performance of masonry construction when subjected to earthquake loads. ASCE 7-98 has been cited here as the appropriate reference for the distribution of seismic forces in order to avoid confusion in the event that the legally adopted building code has no provisions or is inconsistent with the type of distribution upon which these provisions are based.

The special provisions are presented in a cumulative format. Thus the provisions for Seismic Design Categories E and F include provisions for Seismic Design Category D, which include provisions for Seismic Design Category C, and so on.

Seismic requirements for masonry veneers are found in Chapter 6, Veneers.

1.13.2 General

By reference to Section 1.1.3, the designer is permitted to use allowable stress design methods for reinforced masonry, allowable stress design for unreinforced masonry, allowable stress design for prestressed masonry with noted modifications, or empirical design. The alternate method in Section 2.1.3.3 permits a strength design methodology in which allowable stress values are modified to approximate strength value levels. The designer should note that the limitations of the Seismic Design Categories may further limit the available design options. For instance, empirical design procedures are not permitted to be used for structures in Seismic Design Categories D, E, and F. Chapter 5, Empirical Design of Masonry, does not permit empirical design for the lateral force-resisting system in Seismic Design Categories B and C.

If the legally adopted building code has adopted the seismic load provisions of ASCE 7-98, the “strength” design procedures of Section 2.1.3 should be used. If the legally adopted building code has seismic load provisions specifically intended for working stress design, the allowable stress design procedures of Chapter 2 should be used. The architect/engineer should be aware that the use of “strength” level loads should not be used in conjunction with allowable stress design procedures as
overly conservative design can result. Similarly, the use of “allowable stress” loads in conjunction with strength design procedures could result in unconservative designs.

1.13.2.2 Lateral force-resisting system — A lateral force-resisting system must be defined for all buildings. Most masonry buildings use masonry shear walls to serve as the lateral force-resisting system, although other systems are sometimes used (such as concrete or steel frames with masonry infill). Such shear walls must be designed by the engineered methods in Chapter 2, 3, or 4, unless the structure is assigned to Seismic Design Category A, in which case empirical provisions of Chapter 5 may be used.

Five shear wall types are defined, each intended to have a different capacity for inelastic response and energy dissipation in the event of a seismic event. These five shear wall types are assigned different system design parameters such as response modification factors, $R$, based on their expected performance and ductility. Certain shear wall types are permitted in each seismic design category, and unreinforced shear wall types are not permitted in regions of intermediate and high seismic risk. Table 1.13.2 summarizes the requirements of each of the five types of masonry shear walls:

1.13.2.2.1 Ordinary plain (unreinforced) masonry shear walls — These shear walls are permitted to be used only in Seismic Design Categories A and B. Plain masonry walls are designed as unreinforced masonry, although they may in fact contain reinforcement.

1.13.2.2.2 Detailed plain (unreinforced) masonry shear walls — These shear walls are designed as plain (unreinforced) masonry per the sections noted, but contain minimum reinforcement in the horizontal and vertical directions. Because of this reinforcement, these walls have more favorable seismic design parameters, including higher response modification factors, $R$, than ordinary plain (unreinforced) masonry shear walls.

1.13.2.2.2.1 Minimum reinforcement requirements — The provisions of this section require a judgment-based minimum amount of reinforcement to be included in masonry wall construction. Tests reported in Reference 1.17 have confirmed that masonry construction reinforced as indicated performs adequately at this seismic load level. This minimum required reinforcement may also be used to resist design loads.

1.13.2.2.2.2 Connections — Experience has demonstrated that one of the chief causes of failure of masonry construction during earthquakes is inadequate anchorage of masonry walls to floors and roofs. For this reason, an arbitrary minimum anchorage based upon previously established practice has been set. When anchorage is between masonry walls and wood framed floors or roofs, the designer should avoid the use of wood ledgers in cross-grain bending.

<table>
<thead>
<tr>
<th>Shear wall Designation</th>
<th>Design Methods</th>
<th>Reinforcement Requirements</th>
<th>May be Used In</th>
</tr>
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<tr>
<td>Empirical Shear Wall</td>
<td>Section 5.3</td>
<td>None</td>
<td>SDC A</td>
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<td>Section 1.13.2.2.4</td>
<td>SDC A, B &amp; C</td>
</tr>
<tr>
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<td>Section 1.13.2.2.5</td>
<td>SDC A, B, C, D, E &amp; F</td>
</tr>
</tbody>
</table>
1.13.2.2.3 Ordinary reinforced masonry shear walls — These shear walls are required to meet minimum requirements for reinforced masonry as noted in the referenced sections. Because they contain reinforcement, these walls can generally accommodate larger deformations and exhibit higher capacities than similarly configured plain (unreinforced) masonry walls. Hence, they are permitted in both areas of low and moderate seismic risk. Additionally, these walls have more favorable seismic design parameters, including higher response modification factors, \( R \), than plain (unreinforced) masonry shear walls. When assigned to moderate seismic risk areas (Seismic Design Category C), however, minimum reinforcement is required as noted in Section 1.13.2.2.1.

1.13.2.2.4 Intermediate reinforced masonry shear walls — These shear walls are designed as reinforced masonry as noted in the referenced sections, and are also required to contain a minimum amount of prescriptive reinforcement. Because they contain reinforcement, their seismic performance is better than that of plain (unreinforced) masonry shear walls, and they are accordingly permitted in both areas of low and moderate seismic risk. Additionally, these walls have more favorable seismic design parameters including higher response modification factors, \( R \), than plain (unreinforced) masonry shear walls and ordinary reinforced masonry shear walls.

1.13.2.2.5 Special reinforced masonry shear walls — These shear walls are designed as reinforced masonry as noted in the referenced sections and are also required to meet restrictive reinforcement and material requirements. Accordingly, they are permitted to be used in all seismic risk areas. Additionally, these walls have the most favorable seismic design parameters, including the highest response modification factor, \( R \), of any of the masonry shear wall types. The intent of Sections 1.13.2.2.5(a) through 1.13.2.2.5(c) is to provide a minimum level of in-plane shear reinforcement to improve ductility.

1.13.3 Seismic Design Category A

The general requirements of this Code provide for adequate performance of masonry construction in areas of low seismic risk.

1.13.4 Seismic Design Category B

Although masonry may be designed by the provisions of Chapter 2, Allowable Stress Design; Chapter 3, Strength Design of Masonry; Chapter 4, Prestressed Masonry; or Chapter 5, Empirical Design of Masonry, the lateral force-resisting system for structures in Seismic Design Category B must be designed based on a structural analysis in accordance with Chapter 2, 3, or 4. The provisions of Chapter 5 cannot be used to design the lateral force-resisting system of buildings in Seismic Design Category B.

1.13.4.2 Design of elements that are part of the lateral force-resisting system — As a minimum, shear walls in masonry structures assigned to Seismic Design Category B are required to comply with the requirements of ordinary plain (unreinforced), detailed plain (unreinforced), ordinary reinforced, intermediate reinforced, or special reinforced masonry shear walls. Masonry shear walls are required to be designed by either Chapter 2 or Chapter 3 in Seismic Design Categories B and higher.

1.13.5 Seismic Design Category C

In addition to the requirements of Seismic Design Category B, minimum levels of reinforcement and detailing are required. The minimum provisions for improved performance of masonry construction in Seismic Design Category C must be met regardless of the method of design.

1.13.5.3.1 Connections to masonry columns — Experience has demonstrated that connections of structural members to masonry columns are vulnerable to damage during earthquakes unless properly anchored. Requirements are adapted from previously established practice developed as a result of the 1971 San Fernando earthquake.

1.13.5.3.2 Masonry shear walls — Masonry shear walls for structures assigned to SDC C are required to be reinforced because of the increased risk and expected intensity of seismic activity. Ordinary reinforced masonry shear walls, intermediate reinforced masonry shear walls or special reinforced masonry shear walls are required to be used.

1.13.6 Seismic Design Category D

1.13.6.3 Minimum reinforcement requirements for masonry walls — The minimum amount of wall reinforcement has been a long-standing, standard empirical requirement in areas of high seismic loading. It is expressed as a percentage of gross cross-sectional area of the wall. It is intended to improve the ductile behavior of the wall under earthquake loading and assist in crack control. Since the minimum required reinforcement may be used to satisfy design requirements, at least \( 1/3 \) of the minimum amount is reserved for the lesser stressed direction in order to ensure an appropriate distribution in both directions.

1.13.6.4 Masonry shear walls — Masonry shear walls for structures assigned to Seismic Design Category D are required to meet the requirements of special reinforced masonry shear walls because of the increased risk and expected intensity of seismic activity.

1.13.6.5 Minimum reinforcement for masonry columns — Adequate lateral restraint is important for column reinforcement subjected to overturning forces due to earthquakes. Many column failures during earthquakes have been attributed to inadequate lateral tying. For this reason, closer spacing of ties than might
otherwise be required is prudent. An arbitrary minimum spacing has been established through experience. Columns not involved in the lateral force-resisting system should also be more heavily tied at the tops and bottoms for more ductile performance and better resistance to shear.

1.14 — Quality assurance program

The allowable values for masonry design permitted by this Code are valid when the quality of masonry construction meets or exceeds that described in the Specification. Therefore, in order to design masonry by this Code, verification of good quality construction is required. The means by which the quality of construction is monitored is the quality assurance program.

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

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

Section 1.14 establishes the minimum criteria required to assure that the quality of masonry construction conforms to the quality upon which the Code-permissible values are based. The scope of the quality assurance program depends on whether the structure is an essential facility or not, as defined by ASCE 7-98 or the legally adopted building code. Because of their importance, essential facilities are subjected to greater quality assurance measures.

The level of required quality assurance depends on whether the masonry was designed in accordance with Chapters 2, 3, or 4 (engineered) or in accordance with Chapters 5, 6, or 7 (empirical or prescriptive).

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

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

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

Identification and resolution of noncomplying conditions should be addressed in the contract documents. A responsible person should be identified to allow resolution of all nonconformances. In agreement with others in the design/construct team, all resolutions should be either repaired, reworked, accepted as is, or rejected. Repaired and reworked conditions should initiate a reinspection.

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

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

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

Laboratories that comply with the requirements of ASTM C 1093 are more likely to be familiar with masonry materials and testing. Specifying that the testing agencies comply with the requirements of ASTM C 1093 should improve the quality of the resulting masonry.

1.14.7 Acceptance relative to strength requirements

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

1.15 — Construction

The ACI 530.1/ASCE 6/TMS 602 Specification covers material and construction requirements. It is an integral part of the Code in terms of minimum requirements relative to the composition, quality, storage, handling, and placement of materials for masonry structures. The Specification also includes provisions requiring verification that construction achieves the quality specified. The construction must conform to these requirements in order for the Code provisions to be valid.

1.15.1 Grouting, minimum spaces

Code Table 1.15.1 contains the least clear dimension for grouting between wythes and the minimum cell dimensions when grouting hollow units. Selection of units and bonding pattern should be coordinated to achieve these requirements. Vertical alignment of cells must also be considered. All projections or obstructions into the grout space and the diameter of horizontal reinforcement must be considered when calculating the minimum dimensions. See Fig. 1.15-1.

Coarse grout and fine grout are differentiated by aggregate size in ASTM C 476.

The grout space requirements of Code Table 1.15.1 are based on usual grout aggregate size and cleaning practice to permit the complete filling of grout spaces and adequate consolidation using typical methods of construction. Grout spaces smaller than specified in Table 1.15.1 have been used successfully in some areas. When the architect/engineer is requested to accept a grouting procedure that exceeds the limits in Table 1.15.1, construction of a grout demonstration panel is required. Destructive or non-destructive evaluation can confirm that filling and adequate consolidation have been achieved. The architect/engineer should establish criteria for the grout demonstration panel to assure that critical masonry elements included in the construction will be represented in the demonstration panel. Because a single grout demonstration panel erected prior to masonry construction cannot account for all conditions that may be encountered during construction, the architect/engineer should establish inspection procedures to verify grout placement during construction. These inspection procedures should include destructive or non-destructive evaluation to confirm that filling and adequate consolidation have been achieved.

Fig. 1.15-1 — Grout space requirements
1.15.2 Embedded conduits, pipes, and sleeves

1.15.2.1 Conduits, pipes, and sleeves not harmful to mortar and grout can be embedded within the masonry, but the capacity of the wall should not be less than that required by design. The effects of a reduction in section properties in the areas of pipe embedment should be considered. Horizontal pipes located in the planes of walls may affect the wall’s load capacity.

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

Aluminum should not be used in masonry unless it is effectively coated or covered. Aluminum reacts with ions, and may also react electrolytically with steel, causing cracking and/or spalling of the masonry. Aluminum electrical conduits present a special problem since stray electric current accelerates the adverse reaction.

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

References

1.1. ACI Committee 118, “Recommended Documentation for Computer Calculation Submittals to Building Officials,” American Concrete Institute, Farmington Hills, MI.


1.15. ACI Committee 318, “Building Code Requirements for Reinforced Concrete (ACI 318-83),” American Concrete Institute, Detroit, MI 1983, 111 pp.


CHAPTER 2 — ALLOWABLE STRESS DESIGN

2.1 — General

2.1.2 Load combinations

The load combinations were selected by the committee and apply only if the legally adopted building code has none. Nine load combinations are to be considered and the structure designed to resist the maximum stresses resulting from the action of any load combination at any point of the structure. This Code requires that when simultaneous loading is routinely expected, as in the case of dead and live loads, the structure must be designed to fully resist the combined action of the loads prescribed by the legally adopted building code.

2.1.2.3 Previous editions of building codes have customarily used a higher allowable stress when considering wind or earthquake in a structure. This increase has come under attack, and there has been some confusion as to the rationale for permitting the increase. The committee recognizes this situation but has opted to continue to increase allowable stresses in the traditional manner until documentation is available to warrant a change (see Reference 2.1).

2.1.3 Design strength

The structural adequacy of masonry construction requires that the compressive strength of masonry equal or exceed the specified strength. The specified compressive strength $f'_{m}$ on which design is based for each part of the structure must be shown on the project drawings.

2.1.3.3 Strength requirements — The strength of members and connections is based on working stress procedures modified by a factor. The nominal capacity is approximated as the allowable stress increased by $\frac{1}{3}$ (for the load combinations that include wind or earthquake in accordance with Section 2.1.2.3) and further multiplied by a factor of 2.5.

2.1.3.3.1 Required strength — For the initial version of Chapter 4, the use of the same response modification factor ($R$) and the same deflection amplification factor ($C_d$) as for unreinforced masonry will be used. This requirement will ensure that the structural response of prestressed masonry structures designed in accordance with these provisions will essentially remain in the elastic range. When more experimental and field data are available on the ductility of both unbonded and bonded systems, $R$ and $C_d$ factors will be reviewed.

Only part of the reinforcement (nonprestressed) will eventually be replaced by bonded prestressing steel of equal cross sectional area. Unbonded prestressing steel may not be used to replace minimum reinforcement.

2.1.3.3.2 Nominal strength — The resulting nominal strength is approximately 3.3 times the allowable value obtained by using allowable stress design methodology. The design strength is equal to the nominal strength times the strength reduction factor, $\phi$, to achieve a reliable design level value.

Because of the modifications of allowable stress values to strength design levels, some element strengths are calculated using steel stresses in excess of the specified yield. This procedure is correct, and produces designs which are intended to give similar levels of performance as using working stresses in combination with service-level seismic loads.

2.1.4 Anchor bolts solidly grouted in masonry

2.1.4.1 Test design requirements — The design of anchor bolts is based on physical testing. Testing may be used to establish higher working loads than those calculated by Section 2.1.4.2. Many types of anchor bolts, such as expansion anchors, toggle bolts, sleeve anchors, etc., are not included in Section 2.1.4.2 and therefore, such anchors must be designed using test data. ASTM E 448 requires only three tests. The variation in test results for anchors embedded in masonry warrants an increase to the minimum of five stipulated. The variability of anchor bolt strength in masonry and the possibility that anchor bolts may be used in a nonredundant manner results in a safety factor of five.

2.1.4.2 Plate, headed, and bent bar anchor bolts — These design values apply only to the specific bolts mentioned. They are readily available and are depicted in Fig. 2.1-1.

2.1.4.2.1 The minimum embedment depth requirement is considered a practical minimum based on typical construction methods for embedding bolts in masonry. The validity of allowable shear and tension equations for small embedment depths, less than four bolt diameters, has not been verified by tests.

2.1.4.2.2 The results of tests on anchor bolts in tension showed that anchors failed by pullout of a conically shaped section of masonry, or by failure of the anchor itself. Bent bar anchor bolts (J-bolts) often failed by completely sliding out of the specimen. This was due to straightening of the bent end. Eq. (2-1) is the allowable tension load based on masonry failure. The area $A_p$ is the projected area of the assumed failure cone. The cone originates at the bearing point of the embedment and radiates at 45° in the direction of the pull (See Fig. 2.1-2). Comparisons of Eq. (2-1) to test results obtained by Brown and Whitlock show an average factor of safety of approximately eight. Eq. (2-2) is the allowable load for anchor bolts based on failure of the bolt.

The equation allows one-fifth of the yield load for all types of anchor bolts. Eq. (2-1) and (2-2) are plotted in Fig. 2.1-3.
As anchor bolts are spaced closer together, the stresses within the masonry begin to become additive. Therefore, where the spacing between the anchors is less than $2l_b$, this Code requires that the projected areas used to calculate allowable load be reduced to reflect the additive stresses in the area of cone overlap as shown in Fig. 2.1-4.

Test results\(^2\) have shown that the pullout strength of bent bar anchors correlated best with a reduced embedment depth. This may be explained with reference to Fig. 2.1-5. Due to the radius of the bend, stresses are concentrated at a point closer than the full embedment distance.
2.1.4.2.3 Eq. (2-5) was derived from research done by Hatzinikolas et al., and, when compared to tests done by Brown and Whitlock, the factors of safety range from approximately six to eight, respectively. Eq. (2-6) is based on the “shear friction” concept with a coefficient of friction equal to 0.6 and a safety factor of five. Fig. 2.1-6 contains plots of Eq. (2-5) and (2-6).

Sufficient edge distances must be provided such that failures do not occur in modes that are not accounted for in the design equations.

(a) The reason is that with this amount of edge distance, a full failure cone can develop.

(b) The edge distance in the direction of the shear load was derived by equating the following expressions:
Fig. 2.1-5 — Stress distribution on bent anchor bars

Fig. 2.1-6 — Allowable shear stress on anchor bolts
Fig. 2.1-7 — Stress distribution in multiwythe walls of composite masonry

\[ V = \frac{4}{\sqrt{\pi}} f_y \left( \frac{m^2}{2} \right) \] 
(one-half stress cone directed toward free edge)

and

\[ V = 0.6(\pi D^2 / 4) f_y \] 
(anchor steel strength)

This resulted in the following expression:

\[ m = D \sqrt{(0.6 / 8)(f_y / f'_m)} \]

For \( f_y = 60,000 \) psi (413.7 MPa) and \( f'_m = 1,000 \) psi (6.90 MPa), the required edge distance, \( m \), equals 16.4\( D \).

(These equations are for inch-pound units only.)

2.1.4.2.4 Combined shear and tension —

Test results \(^2,^4\) have shown that the strength of anchor bolts follows a circular interaction line. However, for simplicity and additional conservatism, this Code requires a straight line interaction between allowable shear and tension loads.

2.1.5 Multiwythe walls

2.1.5.2 Composite action — Multiwythe walls will act monolithically if sufficient shear strength is developed at the wythe interfaces. See Fig. 2.1-7. Shear transfer is achieved with headers crossing the collar joint or with mortar- or grout-filled collar joints. When mortar- or grout-filled collar joints are relied upon to transfer shear, wall ties are required to ensure structural integrity of the collar joint. Composite action requires that the stresses occurring at the interfaces are within the allowable limits prescribed.

Composite masonry walls generally consist of either brick-to-brick, block-to-block or brick-to-block wythes with the collar joint filled with mortar or grout, and the wythes connected with meal ties. The collar joint thickness ranges from \(\frac{3}{8}\) to 4 in. (9.5 to 102 mm). The joint may contain either vertical or horizontal reinforcement, or reinforcement may be placed in either the brick or block wythe. Composite walls are particularly advantageous for resisting high loads, both in-plane and out-of-plane.

Limited test data \(^2,^4,^5,^6\) are available to document shear strength of collar joints in masonry.

Test results \(^2,^4,^5\) show that shear bond strength of collar joints could vary from as low as 5 psi (34.5 kPa) to as high as 100 psi (690 kPa) depending on type and condition of the interface, consolidation of the joint and type of loading. McCarthy et al. \(^2,^4\) reported an average value of 52 psi (35.9 kPa) with a coefficient of variation of 21.6 percent. A low bound allowable shear value of 5 psi (34.5 kPa) is considered to account for the expected high variability of the interface bond. With some units, Type S mortar slushed collar joints may have better shear bond characteristics than Type N mortar. Results show that thickness of joints, unit absorption and reinforcement have a negligible effect on shear bond strength. Grouted collar joints have higher allowable shear bond stress than
the mortared collar joints. Requirements for masonry headers (Fig. 5.7-1) are empirical and taken from prior codes. The net area of the header should be used in calculating the stress even if a solid unit, which allows up to 25 percent coring, is used. Headers do not provide as much ductility as metal tied wythes with filled collar joints. The influence of differential movement is especially critical when headers are used. The committee does not encourage the use of headers.

A strength analysis has been demonstrated by Porter and Wolde-Tinsae for composite walls subjected to combined in-plane shear and gravity loads. In addition, these authors have shown adequate behavioral characteristics for both brick-to-brick and brick-to-block composite walls with a grouted collar joint. Finite element models for analyzing the interlaminar shearing stresses in collar joints of composite walls have been investigated by Anand et al. They found that
the shear stresses were principally transferred in the upper portion of the wall near the point of load application for the in-plane loads. Thus, below a certain distance, the overall strength of the composite is controlled by the global strength of the wall, providing that the wythes are acting compositely.

The size, number, and spacing of wall ties, shown in Fig. 2.1-8, has been determined from past experience. The limitation of Z-ties to walls of other than hollow units is also based on past experience.

2.1.5.3 Noncomposite action — Multiwythe walls may be constructed so that each wythe is separated from the others by a space which may be crossed only by ties. The ties force compatible lateral deflection, but no composite action exists in the design. Weak axis bending moments caused by either gravity loads or lateral loads are assumed to be distributed to each wythe in proportion to its relative stiffness. See Fig. 2.1-9 for stress distribution in noncomposite walls. Loads due to supported horizontal members are to be carried by the wythe closest to center of span as a result of the deflection of the horizontal member.

The size, number, and spacing of metal ties (Fig. 2.1-8) have been determined from past experience. Ladder-type or tab-type joint reinforcement is required because truss-type joint reinforcement restricts in-plane differential movement between wythes. However, the use of cavity wall ties with drips (bends in ties to prevent moisture migration) has been eliminated because of their reduced load capacity. In cavity walls, this Code limits the thickness of the cavity to 4½ in. (114 mm) to assure adequate performance. If cavity width exceeds 4½ in.
(114 mm), the ties must be designed to carry the loads imposed upon them based on a rational analysis taking into account buckling, tension, pullout, and load distribution.

The NCMA\textsuperscript{2,17} and Canadian Standards Association, CSA\textsuperscript{2,18} have recommendations for use in the design of ties for walls with wide cavities. The term cavity is used when the net thickness is 2 in. (51 mm) or greater. Two in. (51 mm) is considered the minimum space required for resistance to water penetration. A continuous air space of lesser thickness is referred to as a void (unfilled) collar joint. Requirements for adjustable ties are shown in Fig. 2.1-10. They are based on the results in Reference 2.19.

### 2.1.6 Columns

Columns are isolated members usually under axial compressive loads and flexure. If damaged, columns may cause the collapse of other members; sometimes of an entire structure. These critical structural elements warrant the special requirements of this section that were selected after extensive committee consideration.

**2.1.6.1** The minimum nominal side dimension of 8 in. (203 mm) results from practical considerations.

**2.1.6.2** The limit of 25 for the effective height-to-least nominal dimension ratio is based on experience. Data are currently lacking to justify a larger ratio. See Fig. 2.1-11 for effective height determination.

**2.1.6.3** The minimum eccentricity of axial load (Fig. 2.1-12) results from construction imperfections not otherwise anticipated by analysis.

In the event that actual eccentricity exceeds the minimum eccentricity required by this Code, the actual eccentricity should be used. This Code requires that stresses be checked independently about each principal axis of the member (Fig. 2.1-12).

**2.1.6.4** Minimum vertical reinforcement is required in masonry columns to prevent brittle collapse.

The maximum percentage limit in column vertical reinforcement was established based on the committee's experience. Four bars are required so ties can be used to provide a confined core of masonry.

**2.1.6.5** Lateral ties — Lateral reinforcement in columns performs two functions. It provides the required support to prevent buckling of longitudinal column reinforcing bars acting in compression and provides resistance to diagonal tension for columns acting in shear.\textsuperscript{2,20} Ties may be located in the mortar joint.

The requirements of this Code are modeled on those for reinforced concrete columns. Except for permitting \( \frac{1}{4} \) in. (6.4 mm) ties outside of Seismic Design Category D, E, or F, they reflect all applicable provisions of the reinforced concrete code.

### 2.1.7 Pilasters

Pilasters are masonry members which can serve one of several purposes. They may be visible, projecting from one or both sides of the wall, or hidden within the thickness of the wall as shown in Fig. 2.1-13. Pilasters aid in the lateral load resistance of masonry walls and may carry vertical loads.

### 2.1.8 Load transfer at horizontal connections

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

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

![Fig. 2.1-12 — Minimum design eccentricity in columns](image-url)
Fig. 2.1-13 — Typical pilasters
Fig. 2.1-14 — Load distribution

(a) Running Bond

(b) Other than Running Bond

(c) With Bond Beam

Fig. 2.1-15 — Bearing areas

This perimeter of area \( A_2 \) is geometrically similar to and concentric with the bearing area, \( A_1 \).
2.1.9  Concentrated loads

2.1.9.1 Masonry laid in running bond will distribute the axial compressive stress resulting from a concentrated load along the length of wall as described in this Code. Stress can only be transmitted across the head joints of masonry laid in running bond. Thus, when other than running bond is used, concentrated loads can only be spread across the length of one unit unless a bond beam or other technique is used to distribute the load (Fig. 2.1-14).

2.1.9.2 When the supporting masonry area is larger on all sides than the bearing area, this Code allows distribution of concentrated loads over a bearing area \( A_2 \) larger than \( A_1 \), determined as illustrated in Fig. 2.1-15. This is permissible because the confinement of the bearing area by surrounding masonry increases the bearing capacity of the wall in the vicinity of concentrated loads.

2.1.10 Development of reinforcement embedded in grout

2.1.10.1 General — Formulas relative to embedment and splicing have been simplified due to the use of a larger safety factor for masonry than for reinforced concrete.

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

All bars and longitudinal wires must be deformed.

2.1.10.2 Embedment of bars and wires in tension — Eq. (2-8) can be derived from the basic development length expression and an allowable bond stress \( u \) for deformed bars in grout of 160 psi (1103 kPa).\(^{21,22}\) Research\(^{23}\) has shown that epoxy-coated reinforcing bars require longer development length than uncoated reinforcing bars. The 50 percent increase in development length is consistent with ACI 318 provisions.\(^{15}\)
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\[ l_d = d_b F_s / 4u = d_b F_s / 4(160) = 0.0015 d_b F_s \]

\( l_d = 0.22 d_b F_s \) in SI units

2.1.10.3 Embedment of flexural reinforcement — Fig. 2.1-16 illustrates the embedment requirements of flexural reinforcement in a typical continuous beam. Fig. 2.1-17 illustrates the embedment requirements in a typical continuous wall that is not part of the lateral load-resisting system.

2.1.10.3.1.2 Critical sections for a typical continuous beam are indicated with a “c” or an “x” in Fig. 2.1-16. Critical sections for a typical continuous wall are indicated with a “c” in Fig. 2.1-17.

2.1.10.3.1.3 The moment diagrams customarily used in design are approximate. Some shifting of the location of maximum moments may occur due to changes in loading, settlement of supports, lateral loads, or other causes. A diagonal tension crack in a flexural member without stirrups may shift the location of the calculated tensile stress approximately a distance \( d \) toward a point of zero moment. When stirrups are provided, this effect is less severe, although still present.

To provide for shifts in the location of maximum moments, this Code requires the extension of reinforcement a distance \( d \) or \( 12 d_b \) beyond the point at which it is theoretically no longer required to resist flexure, except as noted.

Cutoff points of bars to meet this requirement are illustrated in Fig. 2.1-16.

When bars of different sizes are used, the extension should be in accordance with the diameter of bar being terminated. A bar bent to the far face of a beam and continued there may logically be considered effective in satisfying this section, to the point where the bar crosses the middepth of the member.

2.1.10.3.1.4 Peak stresses exist in the remaining bars wherever adjacent bars are cut off or bent in tension regions. In Fig. 2.1-16 an “x” mark is used to indicate the peak stress points remaining in continuing bars after part of the bars have been cut off. If bars are cut off as short as the moment diagrams allow, these stresses become the full \( F_s \), which requires a full embedment length as indicated. This extension may exceed the length required for flexure.

2.1.10.3.1.5 Evidence of reduced shear strength and loss of ductility when bars are cut off in a tension zone has been reported in Reference 2.24. As a result, this Code does not permit flexural reinforcement to be terminated in a tension zone unless special conditions are satisfied. Flexure cracks tend to open early wherever any reinforcement is terminated in a tension zone. If the stress in the continuing reinforcement and the shear strength are each near their limiting values, diagonal tension cracking tends to develop prematurely from these flexure cracks. Diagonal cracks are less likely

![Fig. 2.1-17 — Development of flexural reinforcement in a typical wall](image-url)
to form where shear stress is low. A lower steel stress reduces the probability of such diagonal cracking.

2.1.10.3.1.6 In corbels, deep flexural members, variable-depth arches, members where the tension reinforcement is not parallel with the compression face, or other instances where the steel stress, \( f_s \), in flexural reinforcement does not vary linearly in proportion to the moment, special means of analysis should be used to determine the peak stress for proper development of the flexural reinforcement.

2.1.10.3.2 Development of positive moment reinforcement — When a flexural member is part of a primary lateral load-resisting system, loads greater than those anticipated in design may cause reversal of moment at supports. As a consequence, some positive reinforcement is required to be anchored into the support. This anchorage assures ductility of response in the event of serious overstress, such as from blast or earthquake. The use of more reinforcement at lower stresses is not sufficient. The full anchorage requirement does not apply to excess reinforcement provided at the support.

2.1.10.3.3 Development of negative moment reinforcement — Negative reinforcement must be properly anchored beyond the support faces by extending the reinforcement \( l_d \) into the support. Other methods of anchoring include the use of a standard hook or suitable mechanical device.

Section 2.1.10.3.3.2 provides for possible shifting of the moment diagram at a point of inflection, as discussed under Commentary Section 2.1.10.3.1.3. This requirement may exceed that of Section 2.1.10.3.1.3 and the more restrictive governs.

2.1.10.4 Hooks

2.1.10.4.1 The allowable stress developed by a standard hook, 7,500 psi (51.7 MPa), is the accepted permissible value in masonry design. Substituting this value into Eq. (2-8) yields the equivalent embedment length given. This value is less than half that given in Reference 1.15.

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

2.1.10.5 Development of shear reinforcement

2.1.10.5.1 Stirrups must be carried as close to the compression face of the member as possible because near ultimate load, flexural tension cracks penetrate deeply.

2.1.10.5.1.2 The requirements for anchorage of U-stirrups for deformed reinforcing bars and deformed wire are illustrated in Fig. 2.1-18.

2.1.10.5.1.2(a) When a standard hook is used, \( 0.5 l_d \) must be provided between \( d/2 \) and the point of tangency of the hook.

This provision may require a reduction in size and spacing of web reinforcement, or an increase in the effective depth of the beam, for web reinforcement to be fully effective.

2.1.10.5.1.3 and 2.1.10.5.1.5 U-stirrups that enclose a longitudinal bar obviously have sufficient resistance in the tension zone of the masonry.

2.1.10.5.2 Welded wire fabric — Although not often used in masonry construction, welded wire fabric provides a convenient means of placing reinforcement in a filled collar joint. See Reference 2.25 for more information.

2.1.10.6 Splices of reinforcement — The importance of continuity in the reinforcement through proper splices is emphasized by the different requirements for the stress level to be transferred in the various types of splices.

2.1.10.6.1 Lap splices — Perhaps the easiest splices to achieve, the length of the splice is based on the allowable stress in the reinforcement.

Fig. 2.1-18 — Anchorage of U-stirrups (deformed reinforcing bars and deformed wire)
The length of lap splices is greater than the required development length of the bars, indicating the assumption of a lower bond stress at the splice.

If individual bars in noncontact lap splices are too widely spaced, an unreinforced section is created, which forces a potential crack to follow a zigzag line. Lap splices may occur with the bars in adjacent grouted cells if the requirements of this section are met.

Welded splices — A full welded splice is primarily intended for large bars (No. 6 [M#19] and larger) in main members. The tensile strength requirement of 125 percent of specified yield strength will ensure sound welding, adequate also for compression. It is desirable that splices be capable of developing the ultimate tensile strength of the bars spliced, but practical limitations make this ideal condition difficult to attain. The maximum reinforcement stress used in design under this Code is based upon yield strength. To ensure sufficient strength in splices so that brittle failure can be avoided, the 25 percent increase above the specified yield strength was selected as both an adequate minimum for safety and a practicable maximum for economy.

Mechanical connections — Full mechanical connections are also required to develop 125 percent of the yield strength in tension or compression as required, for the same reasons discussed for full welded splices.

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

Unreinforced masonry

This section provides for the design of masonry members in which tensile stresses, not exceeding allowable limits, are resisted by the masonry. This has previously been referred to as unreinforced or plain masonry. Flexural tensile stresses may result from bending moments, from eccentric vertical loads, or from lateral loads.

A fundamental premise is that under the effects of design loads, masonry remains uncracked. One must be aware, however, that stresses due to restraint against differential movement, temperature change, moisture expansion, and shrinkage combine with the design load stresses. Stresses due to restraint should be controlled by joints or other construction techniques to ensure that the combined stresses do not exceed the allowable.

Stresses in reinforcement

Reinforcement may be placed in masonry walls to control the effects of movements from temperature changes or shrinkage.

Axial compression and flexure

For a member solely subjected to axial load, the resulting compressive stress $f_a$ should not exceed the allowable compressive stress $F_a$; in other words, $f_a/F_a$ should not exceed 1. Similarly, in a member subjected solely to bending, the resulting compressive stress $f_b$ in the extreme compression fiber should not exceed the allowable compressive stress $F_b$, or again, $f_b/F_b$ should not exceed 1.

This Code requires that under combined axial and flexure loads, the sum of the quotients of the resulting compression stresses to the allowable $(f_a/F_a + f_b/F_b)$ does not exceed 1. This unity interaction equation is a simple portioning of the available allowable stresses to the applied loads, and is used to design masonry for compressive stresses. The unity formula can be extended when biaxial bending is present by replacing the bending stress quotients with the quotients of the calculated bending stress over the allowable bending stress for both axes.

In this interaction equation, secondary bending effects resulting from the axial load are ignored. A more accurate equation would include the use of a moment magnifier applied to the flexure term, $f_b/F_b$. Although avoidance of a moment magnifier term will produce nonconservative results, the committee decided not to include this term in Eq. (2-10) for the following reasons:

- At larger $h/r$ values, where moment magnification is more critical, the allowable axial load on the member will be limited by Code Eq. (2-11).
- For the practical range of $h/r$ values, errors induced by ignoring the moment magnifier will be relatively small, less than 15 percent.
- The overall safety factor of 4 included in the allowable stress equations is sufficiently large to allow this simplification in the design procedure.

The requirement of Eq. (2-11) that the axial compressive load $P$ not exceed $1/4$ of the buckling load $P_e$ replaces the arbitrary upper limits on slenderness used in ACI 531.27.

The purpose of Eq. (2-11) is to safeguard against a premature stability failure caused by eccentrically applied axial load. The equation is not intended to be used to check adequacy for combined axial compression and flexure. Therefore, in Eq. (2-15), the value of the eccentricity “$e$” that is to be used to calculate $P_e$ is the actual eccentricity of the applied compressive load. The value of “$e$” is not to be calculated as $M_{max}$ divided by $P$ where $M_{max}$ is a moment caused by other than eccentric load.
Eq. (2-11) is an essential check since the allowable compressive stress for members with an $h/r$ ratio in excess of 99 has been developed assuming only a nominal eccentricity of the compressive load. Thus, when the eccentricity of the compressive load exceeds the minimum eccentricity of 0.1$t$, Eq. (2-13) will overestimate the allowable compressive stress and Eq. (2-11) may control.

The allowable stress values for $F_a$, presented in Eqs. (2-12) and (2-13) are based on an analysis of the results of axial load tests performed on clay and concrete masonry elements. A fit of an empirical curve to this test data, Fig. 2.2-1, indicates that members having an $h/r$ ratio not exceeding 99 fail under loads below the Euler buckling load at a stress level equal to:

$$f'_m \left[1 - (h / 140r)^2 \right] \quad \text{(equation unchanged with SI units)}$$

Thus, for members having an $h/r$ ratio not exceeding 99, this Code allows axial load stresses not exceeding $1/4$ of the aforementioned failure stress.

Applying the Euler theory of buckling to members having resistance in compression but not in tension, References 2.28, 2.29, and 2.30 show that for a solid section, the critical compressive load for these members can be expressed by the formula

$$P_e = \left(\pi^2 E_m I_n / h^2\right) \left(1 - 2e / t\right) \left(1 - 0.577 \left(\frac{e}{t}\right)^3\right)$$

(2-15)

in which

$$I_n = \text{uncracked moment of inertia}$$

$e = \text{eccentricity of axial compressive load with respect to the member longitudinal centroidal axis.}$

In the derivation of this buckling load equation, tension cracking is assumed to occur prior to failure.

For $h/r$ values in excess of 99, the limited test data is approximated by the buckling load.

For a solid rectangular section, $r = \sqrt{t^2 / 12}$. Making this substitution into the buckling load equation gives

$$P_e = \left(\pi^2 E_m I_n / h^2\right) \left(1 - 0.577 \left(\frac{e}{t}\right)^3\right)$$

(2-15)

Transforming the buckling equation using a minimum eccentricity of 0.1$t$ (from Section 2.1.6.3) and an elastic modulus equal to 1000 $f'_m$, the axial compressive stress at buckling failure amounts approximately to $\left[70(r / h)^2 f'_m\right]$. Thus, for members having an $h/r$ ratio in excess of 99, this Code allows an axial load compressive stress not exceeding $1/4$ of this failure stress [Eq. (2-13)].

Flexure tests of masonry to failure have shown that the compressive stress at failure computed by the straight line theory exceeds that of masonry failing under axial load. This phenomenon is attributed to the restraining effect of less highly strained compressive fibers on the fibers of maximum compressive strain. This effect is less pronounced in hollow masonry than solid masonry; however, the test data indicate that, computed by the straight line theory,
the compressive stress at failure in hollow masonry subjected to flexure exceeds by \( \frac{1}{3} \) that of the masonry under axial load. Thus, to maintain a factor of safety of 4 in design, the committee considered it conservative to establish the allowable compressive stress in flexure as:

\[
f_b = \gamma_b \times (\frac{1}{3}) f_m = (\gamma_b) f_m'
\]

### 2.2.3.2 Allowable flexural tensile stresses for portland-cement lime mortar are traditional values.

Mortar cement is a product that has bond strength requirements which have been established to provide comparable flexural tensile bond strength to that achieved using portland cement-lime mortar.\(^2\)\(^3\),\(^4\)\(^5\),\(^6\)

For masonry cement and air entrained portland-cement lime mortar, there are no conclusive research data and, hence, flexural tensile stresses are based on existing requirements in other codes.

The tensile stresses listed are only for tension due to flexure under out-of-plane loading. Flexural tensile stresses can be offset by axial compressive stress, but the resultant tensile stress due to combined bending and axial compression cannot exceed the allowable flexural tensile stress. Note, no values for allowable tensile stress are given in this Code for in-plane bending because flexural tension from in-plane bending in walls should be carried by reinforcement.

Variables affecting tensile bond strength of brick masonry normal to bed joints include mortar properties, unit initial rate of absorption, surface condition, workmanship and curing condition. For tension parallel to bed joints, the strength and geometry of the units will also have an effect on tensile strength.

Test data using a bond wrench\(^2\)\(^3\),\(^8\)\(^9\) revealed tensile bond strength normal to bed joints ranging from 30 psi (207 kPa) to 190 psi (1,310 kPa). This wide range is attributed to the multitude of parameters affecting tensile bond strength.

Test results\(^2\)\(^9\),\(^10\) show that masonry cement mortars and mortars with high air content generally have lower bond strength than portland cement-lime mortars.

Tests conducted by Hamid\(^2\)\(^1\) show the significant effect of the aspect ratio (height to least dimension) of the brick unit on the flexural tensile strength. The increase in the aspect ratio of the unit results in an increase in strength parallel to bed joints and a decrease in strength normal to bed joints.

Research work\(^2\)\(^2\) on flexural strength of concrete masonry has shown that grouting has a significant effect in increasing strength capacity over ungrouted masonry. A three-fold increase in tensile strength normal to bed joints was achieved using fine grout as compared to ungrouted masonry. The results also show that, within a practical range of strength, the actual strength of grout is not of major importance. For tension parallel to bed joints, a 133 percent increase in flexural strength was achieved by grouting all cells. Grout cores change the failure mode from stepped-wise cracking along the bed and head joints for hollow walls to a straight line path along the head joints and unit for grouted walls.

Research\(^2\)\(^3\) has shown that flexural strength of unreinforced grouted concrete and clay masonry is largely independent of mortar type or cementitious materials.

For partial grouting, the footnote permits interpolation between the fully grouted value and the hollow unit value based on the percentage of grouting. A concrete masonry wall with Type S portland cement-lime mortar grouted 50 percent and stressed normal to the bed joints would have an allowable stress midway between 65 psi (448 kPa) and 25 psi (172 kPa), hence an allowable stress of 45 psi (310 kPa).

### 2.2.4 Axial tension

Tensile stresses in masonry walls due to axially applied load are not permitted. If axial tension develops in walls due to uplift of connected roofs or floors, the walls must be reinforced to resist the tension. Cumulative compressive stress from dead load can be used to offset axial tension. Masonry columns are required to have vertical reinforcing by Section 2.1.6.4.

### 2.2.5 Shear

Three modes of shear failure in unreinforced masonry are possible:

(a) Diagonal tension cracks form through the mortar and masonry units.

(b) Sliding occurs along a straight crack at horizontal bed joints.

(c) Stepped cracks form, alternating from head joint to bed joint.

In the absence of suitable research data, the committee recommends that the allowable shear stress values given in Code Section 2.2.5.2 be used for limiting out-of-plane shear stresses.

#### 2.2.5.1 The theoretical parabolic stress distribution is used to calculate shear stress rather than the average stress. Many other codes use average shear stress so direct comparison of allowable values is not valid. Effective area requirements are given in Section 1.9.1. For rectangular sections this equates to \( \frac{1}{2} V/A \).

This equation is also used to calculate shear stresses for composite action.

#### 2.2.5.2 Shear stress allowable values are applicable to shear walls without reinforcement. The values given are based on recent research.\(^2\)\(^4\)\(^5\)\(^6\) The 0.45 coefficient of friction, increased from 0.20, is shown in these tests. \( N_v \) is normally based on dead load.
For masonry in other than running bond with bond beam spaced less than or equal to 48 in. (1219 mm) and running bond masonry, \( b \) equals the lesser of:

\[
\begin{align*}
  b &= s \\
  b &= 6t \\
  b &= 72 \text{ in. (1829 mm)}
\end{align*}
\]

For masonry in other than running bond with bond beams spaced greater than 48 in. (1219 mm), \( b \) equals the lesser of:

\[
\begin{align*}
  b &= s \\
  b &= \text{length of unit}
\end{align*}
\]

*Fig. 2.3-1 — Width of compression area*

### 2.3 — Reinforced masonry

#### 2.3.1 Scope

The requirements covered in this section pertain to the design of masonry previously referred to as “reinforced masonry.” The term, reinforced masonry, has been avoided to more accurately describe the conditions of design covered in Sections 2.2 and 2.3. Additionally, it will avoid confusion with masonry designed in accordance with the provisions of Section 2.2 in which the effect of joint and other reinforcement used in construction is neglected in the design.

Tension still develops in the masonry, but it is not considered to be effective in resisting design loads.

#### 2.3.2 Steel reinforcement - Allowable stresses — These values have been in use for many years.

#### 2.3.3 Axial compression and flexure

See commentary for 2.2.3.1.

##### 2.3.3.2 Allowable forces and stresses — This Code limits the compressive stress in masonry members based on the type of load acting on the member. The compressive force at the section resulting from axial loads or from the axial component of combined loads is calculated separately, and is limited to the values permitted in Section 2.3.3.2.1. Equation (2-17) or (2-18) will control the capacity of columns with large axial loads. The coefficient of 0.25 provides a factor of safety of about 4.0 against crushing of masonry. The coefficient of 0.65 was determined from tests of reinforced masonry columns and is taken from previous masonry codes. A second compressive stress calculation must be performed considering the combined effects of the axial load component and flexure at the section and should be limited to the values permitted in Section 2.3.3.2.2. (See commentary for Section 2.2.3.)

##### 2.3.3.3 Effective compressive width per bar —

The effective width of the compressive area for each reinforcing bar must be established. Fig. 2.3-1 depicts the limits for the conditions stated. Limited research is available on this subject.

The limited ability of head joints to transfer stress when the masonry is laid in stack bond is recognized by the requirements for bond beams. Masonry units with open ends that are solidly grouted will transfer stress as indicated in Section 2.2.5.2(d) and can qualify as running bond.

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

##### 2.3.3.4 Beams —

The requirements for masonry members outlined are relatively straightforward, and follow generally accepted engineering practice.

The minimum bearing length of 4 in. (102 mm) in the direction of span is considered a reasonable minimum for masonry beams over door and window openings to prevent concentrated compressive stresses at the edge of the opening. This requirement should also apply to beams and lintels in the plane of the wall.

To minimize lateral torsional buckling, Section 2.3.3.4.4 requires lateral bracing of the compression face in accordance with standard limits for beams of other materials. The requirement applies to simply supported beams as written. With continuous or fixed beams, the spacing may be increased.
2.3.5 Shear

To compensate for a simplified method of analysis and unknowns in construction, the shear stresses allowed by this Code are conservative. When reinforcement is added to masonry, the shear resistance of the element is increased. Priestley and Bridgemen\textsuperscript{250} concluded from a series of tests that shear reinforcement is effective in providing resistance only if it is designed to carry the full shear load. Thus, most codes do not add the shear resistance provided by the masonry to that provided by the steel. The amount of design shear reinforcement is specified to resist one hundred percent of the applied shearing load. See Commentary Section 2.2.5 and the flow chart for design of masonry members resisting shear shown in Fig. 2.3-2.

2.3.5.2 Eqs. (2-19) through (2-25) in Code Section 2.3.5.2 are derived from previous masonry codes.\textsuperscript{227,251,252}

2.3.5.2.1 Shear forces can act both vertically and horizontally under wind and seismic conditions in shear walls. Since the beams are reinforced and will exhibit flexural cracking, the classical shear stress calculation used in Section 2.2 is replaced with an approximation of the maximum shear stress below the neutral axis. The approximation results from deleting the term “j” in the equation $f_r = V/hjd$. 

---

**Fig. 2.3-2 — Flow chart for shear design**
2.3.5.2.3(a) The limits on the calculated shear stress in beams are in conformance with those found in previous masonry codes.

2.3.5.3 Eq. (2-26) may be derived by assuming a 45 degree shear crack extended from the extreme compression fiber to the centroid of the tension steel, which is the distance $d$. Forces are summed in the direction of the shear reinforcement and the doweling resistance of the longitudinal reinforcement is neglected. In Eq. (2-26), for shear walls without shear reinforcement and for shear parallel to the plane of the wall, $d_v$ may be substituted for $d$. Notice that for such shear walls, $d_v$ may be either horizontal or vertical, depending on the direction of the shear and resulting reinforcement.

For shear walls, the longitudinal reinforcement is normally vertical and distributed along the length of the wall. The shear reinforcement is normally horizontal. In the development of the equation for shear walls, the 45 degree crack extends through more horizontal reinforcement than that obtained by using the depth to the centroid of the steel $d$. Thus, the use of $d_v$ is justified. However, the designer must be cautioned that this is not always the case. For example, in a 10 ft (3.05 m) shear wall with vertical reinforcement located 2 ft (0.61 m) from each end (with no other vertical reinforcement) it would be unconservative to use $d_v$ and the maximum reinforced length may be used in place of $d_v$.

2.3.5.3.1 The assumed shear crack is at 45 degrees to the longitudinal reinforcement. Thus, a maximum spacing of $d/2$ is specified to assure that each crack is crossed by at least one bar. The 48 in. (1219 mm) maximum spacing is an arbitrary choice which has been in codes for many years.

2.3.5.4 Shear across collar joints in composite masonry walls is transferred by the mortar or grout in the collar joint. Shear stress in the collar joint or at the interface between the wythe and the collar joint is limited to the allowable stresses in Section 2.1.5.2.2. Shear transfer by wall ties or other reinforcement across the collar joint is not considered.

2.3.5.5 The beam or wall loading within $d/2$ of the support is assumed to be carried in direct compression or tension to the support without increasing the shear load, provided no concentrated load occurs within the $d/2$ distance.

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CHAPTER 3 — STRENGTH DESIGN OF MASONRY

3.1.3 Design strength

3.1.3.1 Seismic design provisions — In order to accurately distribute loads in a structure subjected to lateral loading, the lateral stiffness of all structural members should be considered. Although structures may be designed to use shear walls for lateral-load resistance, columns may also be incorporated for vertical capacity. The stipulation that lateral load resisting elements provide at least 80 percent of the lateral stiffness helps ensure that additional elements do not significantly contribute to the lateral stiffness. Based on typical design assumptions, the lateral stiffness of structural elements should be based on cracked section properties for reinforced masonry and uncracked section properties for unreinforced masonry.

3.1.4 Strength reduction factors

The strength reduction factor incorporates the difference between the nominal strength provided in accordance with the provisions of Chapter 3 and the expected strength of the as-built masonry. The strength reduction factor also accounts for the uncertainties in construction, material properties, calculated versus actual member strengths, as well as anticipated mode of failure.

3.1.4.1 Combinations of flexure and axial load in reinforced masonry — The same strength reduction factor is used for the axial load and the flexural tension or compression induced by bending moment in reinforced masonry elements. The higher strength reduction factor associated with reinforced elements (in comparison to unreinforced elements) reflects a decrease in the coefficient of variation of the measured strengths of reinforced elements when compared to similarly configured unreinforced elements.

3.1.4.2 Combinations of flexure and axial load in unreinforced masonry — The same strength reduction factor is used for the axial load and the flexural tension or compression induced by bending moment in unreinforced masonry elements. The lower strength reduction factor associated with unreinforced elements (in comparison to reinforced elements) reflects an increase in the coefficient of variation of the measured strengths of unreinforced elements when compared to similarly configured reinforced elements.

3.1.4.3 Shear — Strength reduction factors for calculating the design shear strength are commonly more conservative than those associated with the design flexural strength. However, the capacity design provisions of Chapter 3 require that shear capacity considerably exceed flexural capacity. Hence, the strength reduction factor for shear is taken as 0.80, a value 33 percent larger than the historical value.

3.1.4.4 Anchor bolts — Because of the general similarity between the behavior of anchors embedded in grout and in concrete, and because available research data for anchors in grout indicate similarity, the strength reduction values associated with various controlling anchor bolt failures are derived from expressions based on research into the performance of anchors embedded in concrete.

3.1.4.6 Bearing — The value of the strength reduction factor used in bearing assumes that some degradation has occurred within the masonry material.

3.1.5 Deformation requirements

3.1.5.1 Drift limits — This section provides procedures for the limitation of story drift. The term "drift" has two connotations:

1. "Story drift" is the maximum calculated lateral displacement within a story (that is, the calculated displacement of one level relative to the level below caused by the effects of design seismic loads).

2. The calculated lateral displacement or deflection due to design seismic loads is the absolute displacement of any point in the structure relative to the base. This is not "story drift" and is not to be used for drift control or stability considerations since it may give a false impression of the effects in critical stories. However, it is important when considering seismic separation requirements.

Overall or total drift is the lateral displacement of the top of a building relative to the base. The overall drift ratio is the total drift divided by the building height. Story drift is the lateral displacement of one story relative to an adjacent story. The story drift ratio is the story drift divided by the corresponding story height. The overall drift ratio is usually an indication of moments in a structure and is also related to seismic separation demands. The story drift ratio is an indication of local seismic deformation, which relates to seismic separation demands. The maximum story drift ratio could exceed the overall drift ratio.

There are many reasons for controlling drift; one is to control member inelastic strain. Although the relationship between lateral drift and maximum strain is imprecise, so is the current state of knowledge of what strain limitations should be.

Under small lateral deformations, secondary stresses are normally within tolerable limits. However, larger deformations with heavy vertical loads can lead to significant secondary moments from P-delta effects in the design. The drift limits indirectly provide upper bounds for these effects.

Buildings subjected to earthquakes need drift control to restrict damage to partitions, shaft and stair enclosures, glass, and other fragile nonstructural elements and, more importantly, to minimize differential movement demands on the seismic-force-resisting elements.
The designer must keep in mind that the drift limits, \( \Delta_n \), correspond to story drifts and, therefore, are applicable to each story (that is, they must not be exceeded in any story even though the drift in other stories may be well below the limit).

Although the provisions of this Code do not give equations for computing building separations, the distance should be sufficient to avoid damaging contact under total calculated deflection for the design loading in order to avoid interference and possible destructive hammering between buildings. The distance should be equal to the total of the lateral deflections of the two units assumed deflecting toward each other (this involves increasing the separation with height). If the effects of hammering can be shown not to be detrimental, these distances may be reduced. For very rigid shear wall structures with rigid diaphragms whose lateral deflections are difficult to estimate, older code requirements for structural separations of at least 1 in. (25.4 mm) plus 1/2 in. (12.7 mm) for each 10 ft (3.1 m) of height above 20 ft (6.1 m) should be followed.

**3.1.5.2 Deflection of unreinforced (plain) masonry** — The deflection calculations of unreinforced masonry are based on elastic performance of the masonry assemblage as outlined in the design criteria of Section 3.3.1.3.

**3.1.5.3 Deflection of reinforced masonry** — Values of \( I_{eg} \) are typically about one-half of \( I_g \) for common solid grouted element configurations. Calculating a more accurate effective moment of inertia using a moment curvature analysis may be desirable for some circumstances. Historically, an effective moment of inertia has been calculated using net cross-sectional area properties and the ratio of the cracking moment strength based on appropriate modulus of rupture values to the applied moment resulting from unfactored loads as shown in the following equation. This equation has successfully been used for estimating the post-cracking flexural stiffness of both concrete and masonry.\(^{3.10}\)

\[
I_{eff} = I_n \left( \frac{M_{cr}}{M_a} \right)^3 + I_{cr} \left[ 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right] \leq I_n \leq 0.5I_g
\]

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

The design equations provided in the Code stem from research conducted on cast-in-place headed anchor bolts and bent-bar anchors (J- or L-bolts) in grout. Therefore, the application of these provisions to post-installed anchors may be in question. Due to the wide variation in configurations of post-installed anchors, designers are referred to product literature published by the manufacturers.

**3.1.6.1 Nominal axial tensile strength of headed anchor bolts** — The tensile strength of a headed anchor bolt is governed by yield and fracture of the anchor steel or by breakout of an approximately conical volume of masonry starting at the anchor head and having a fracture surface oriented at approximately 45 degrees to the masonry surface. (Refer to Commentary Fig. 2.1-2.) Steel strength is calculated conventionally using the effective tensile stress area of the anchor (that is, including the reduction in area of the anchor shank due to threads).

**3.1.6.2 Nominal axial tensile strength of bent-bar anchor bolts** — The tensile strength of a bent-bar anchor bolt (J- or L-bolt) is governed by yield and fracture of the anchor steel, by tensile cone breakout of the masonry, or by straightening and pullout of the anchor from the masonry. Capacities corresponding to the first two failure modes are calculated as for headed anchor bolts. The Code requires that when the second term of Eq. (3-6) is used in the design calculations, precautions are taken to ensure that the shanks of the bent-bar anchors are clean and free of foreign debris that would otherwise interfere with the bond between the bolt and the grout.

**3.1.6.3 Nominal shear strength of headed and bent-bar anchor bolts** — The shear strength of a headed or bent-bar anchor bolt is governed by yield and fracture of the anchor steel or by masonry shear breakout. Steel strength is calculated conventionally using the effective tensile stress area (that is, threads are conservatively assumed to lie in the critical shear plane). Under static shear loading, bent-bar anchor bolts (J- or L-bolts) do not exhibit straightening and pullout. Under reversed cyclic shear however, available research\(^{3.1}\) suggests that straightening and pullout may occur.

**3.1.7 Material properties**

Refer to Commentary Section 1.8 for additional information.

**3.1.7.1 Compressive strength**

**3.1.7.1.1 Masonry compressive strength** — Research\(^{3.2}\) has been conducted on structural masonry components having a compressive strength in the range of 1,500 to 6,000 psi (10.34 to 41.37 MPa). Design criteria are based on these research results. Design values therefore are limited to compressive strengths in the range of 1,500 to 4,000 psi (10.34 to 27.58 MPa) for concrete masonry and 1,500 to 6,000 psi (10.34 to 41.37 MPa) for clay masonry.

**3.1.7.1.2 Grout compressive strength** —

Since most empirically derived design equations relate the calculated nominal strength as a function of the specified compressive strength of the masonry, the specified compressive strength of the grout is required to be at least equal to the specified compressive strength for concrete masonry. This requirement is stipulated to ensure that where the grout compressive strength may significantly control the design (such as anchors embedded in grout), the nominal strength will not be affected. The limitation on the maximum grout...
compressive strength is due to the lack of available research investigating material strengths beyond this boundary.

3.1.7.2 Masonry modulus of rupture

3.1.7.2.1 Out-of-plane bending — The modulus of rupture values provided in Code Table 3.1.7.2.1 are intended to be directly related to the allowable stress values for flexural tension multiplied by an appropriate factor to approximate the nominal strength values.

3.1.7.2.2 In-plane bending — The value for the modulus of rupture for running bond masonry bending in-plane is empirically derived from research data. Stack bond masonry has historically been assumed to have no flexural bond strength across mortared head joints; thus, the grout area alone is used.

3.1.7.3 Reinforcement strength — Research conducted on reinforced masonry components used Grade 60 steel. To be consistent with laboratory documented investigations; design is based on a nominal steel yield strength of 60,000 psi (413.7 MPa). The limitation on the steel yield strength of 130 percent of the nominal yield strength is to minimize the over-strength unintentionally incorporated into a design.

3.2 — Reinforced masonry

3.2.1 Scope

Reinforcement complements the high compressive strength of masonry with high tensile strength. Increased load-carrying capacity and greater ductility result from the use of reinforcement in the design of masonry structures.

3.2.2 Design assumptions

The design principles listed are those that traditionally have been used for reinforced masonry members.

The values for the maximum usable strain are based on research on masonry materials. Concern has been raised as to the implied precision of the values. However, the Committee agrees that the reported values for the maximum usable strain reasonably represent those observed during testing.

While tension may still develop in the masonry of a reinforced element, it is not considered effective in resisting design loads, but is considered to contribute to the overall stiffness of a masonry element.

3.2.3 Reinforcement requirements and details

3.2.3.1 Reinforcing bar size limitations — The limit of using a No. 9 (M #29) bar is motivated by the desire to use a larger number of smaller diameter bars to transfer stresses rather than a fewer number of larger diameter bars. Some research investigations have concluded that in certain applications masonry reinforced with more uniformly distributed smaller diameter bars performs better than similarly configured masonry elements using fewer larger diameter bars. While not all investigations are conclusive, the Committee does agree that incorporating larger diameter reinforcement may dictate unreasonable cover distances or development lengths. The limitations on clear spacing and percentage of cell area are indirect methods of preventing problems associated with over-reinforcing and grout consolidation. At sections containing lap splices, the maximum area of reinforcement should not exceed 8 percent of the cell area.

3.2.3.2 Standard hooks — Refer to Commentary Section 1.12.5 for further information.

3.2.3.3 Development — The clear spacing between adjacent reinforcement does not apply to the reinforcing bars being spliced together. Refer to Commentary 3.2.3.4 for further information.

3.2.3.3.1.1 The edge vertical bar is the last reinforcing bar in walls without intersecting walls, and is the bar at the intersection of walls that intersect. Hooking the horizontal reinforcement around a vertical bar located within the wall running parallel to the horizontal reinforcement would cause the reinforcement to protrude from the wall.

3.2.3.4 Splices — The required length of the lap splice is based on developing a minimum reinforcing steel stress of 1.25f_y. This requirement provides adequate capacity while maintaining consistent requirements between lap, mechanical, and welded splices. Historically, the length of lap has been based on the bond stress that is capable of being developed between the reinforcing steel and the surrounding grout. Testing has shown that bond stress failure (or pull-out of the reinforcing steel) is only one possible mode of failure for lap splices. Other failure modes include rupture of the reinforcing steel and longitudinal splitting of masonry along the length of the lap. Experimental results of several independent research programs were combined and analyzed to provide insight into predicting the necessary lap lengths for reinforcement splices in masonry construction.

To develop a reasonable design formula, multiple regression analysis was used to find the form of a good predictive model. The following equation yielded the best prediction of measured capacities of the tested splices:

\[
T_r = -176240 + 3053l_c + 252043d_b^2 + 3217\sqrt{f_{mt}} + 33317c_{cl}
\]

Where:

- \( T_r \) = predicted tensile strength of the splice, lb (N);
- \( l_c \) = tested length of lap splice, in. (mm);
- \( f_{mt} \) = tested compressive strength of masonry, psi (MPa); and
- \( c_{cl} \) = clear cover of structural reinforcement, in. (mm).
The square of the Pearson product moment correlation coefficient of this equation is 0.932, showing excellent correlation between the measured and predicted strength of the splices. Figure 3.2-1 graphically shows the equation predictions compared to results of the individual test programs.

Next, by replacing the predicted strength of the splice with \(1.25A_b f_y\) (imposing the same requirement on lap splices as required for mechanical and welded splices) and solving for the resulting splice length yields:

\[
l_s = \frac{1.25A_b f_y + 176240 - 25204.3d_b^2 - 321.7f_m^{0.5} - 3331.7c_{cl}}{305.3}
\]

Since the form of this equation is impractical for everyday design applications, Code equation (3-14) was fitted to the equation shown above.

### 3.2.3.5 Maximum reinforcement percentages –

The goal of this provision is to provide ductile response, for example, by increasing inelastic rotation capacity potential hinging regions. To help achieve this goal, tensile reinforcement is limited so that the compressive zone of the member will not crush before the tensile reinforcement develops the inelastic strain consistent with the maximum drift limits of Section 3.1.5. For all masonry components other than walls bending in the out-of-plane direction, maximum reinforcement is limited in accordance with a prescribed strain distribution based on a tensile strain equal to five times the yield strain for the reinforcing bar closest to the edge of the member, and a maximum masonry compressive strain equal to 0.0025 for concrete masonry or 0.0035 for clay masonry. By limiting longitudinal reinforcement in this manner, inelastic curvature capacity is easily depicted as the slope of this strain distribution.

Because axial force is implicitly considered in the determination of maximum longitudinal reinforcement, inelastic curvature capacity can be relied upon independent of the level of axial compressive force. Thus, the capacity reduction factors for axial load and flexure can be the same as for flexure alone. Also, confinement reinforcement is not required because the maximum masonry compressive strain will be less than the ultimate values.

Calculated tensile force in the reinforcement is based on a stress equal to 1.25 times the yield stress to account for differences between the actual yield strength and the minimum specified strength, and the possibility of strain hardening. This increase of stress beyond yield also compensates for effects of discontinuous tensile strain fields that develop as a result of tensile cracking.

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**Fig. 3.2-1 — Relationship between measured and predicted splice capacities**
The required out-of-plane strain limits need not be as high as that for in-plane limits, because the inelastic curvature is developed over a greater portion of the member length. Finally, out-of-plane requirements were refined so that the maximum reinforcement limits as governed by out-of-plane requirements would be approximately the same as those governed by in-plane requirements, for typical wall geometries.

For walls bending out-of-plane, the limit on maximum reinforcement is relaxed by considering a strain distribution based on 1.3 times the yield strain for the reinforcing bar closest to the member edge. This limiting strain distribution is less severe than that adopted for in-plane bending.

3.2.3.5.1 Using the critical strain conditions of 3.2.3.5.1(a), compatibility, stress-strain relationships, and equilibrium, the maximum reinforcement limits for these elements can be derived. The same holds true for Section 3.2.3.5.1(b) for walls loaded out-of-plane.

3.2.3.5.2 Using the critical strain conditions of 3.2.3.5.2, compatibility, stress-strain relationships, and equilibrium, the maximum reinforcement limits for these elements can be derived. The unfactored gravity axial loads referred to in this provision are intended to be the gravity components of the allowable stress design loading combinations that include earthquake from the legally adopted building code.

3.2.3.6 Bundling of reinforcing bars — This requirement stems from the lack of research on masonry with bundled bars.

3.2.4 Design of beams, piers, and columns

3.2.4.1 Nominal strength

3.2.4.1.1 Nominal axial and flexural strength — The nominal flexural strength of a member may be calculated using the assumption of an equivalent rectangular stress block as outlined in Section 3.2.2. Refer to Commentary Section 2.2.3 for further information regarding slenderness effects on axial load capacity as taken into account with the use of Eq. (3-16) and Eq. (3-17), Eq. (3-16) and Eq. (3-17) apply to simply supported end conditions and transverse loading which results in a symmetric deflection (curvature) about the midheight of the element, if present. Where other support conditions or loading scenarios are known to exist, Eq. (3-16) and Eq. (3-17) should be modified accordingly to account for the effective height of the element or shape of the bending moment diagram over the clear span of the element. The first coefficient, 0.80, in Eq. (3-16) and Eq. (3-17) accounts for unavoidable minimum eccentricity in the axial load.

3.2.4.1.2 Nominal shear strength — The limitations on maximum nominal shear strength are included to preclude critical (brittle) shear-related failures.

3.2.4.1.2.1 Nominal masonry shear strength — Eq. (3-21) is empirically derived from research.

3.2.4.1.2.2 Nominal shear strength provided by reinforcement — Eq. (3-22) is empirically derived from research.

3.2.4.2 Beams — This section applies to the design of lintels and beams.

3.2.4.2.2 Longitudinal reinforcement

3.2.4.2.2.1 Restricting the variation of bar sizes in a beam is included to increase the depth of the member compression zone and to increase member ductility. When incorporating two bars of significantly different sizes in a single beam, the larger bar requires a much higher load to reach yield strain, in effect “stiffening” the beam.

3.2.4.2.2.2 The requirement that the nominal flexural strength of a beam not be less than 1.3 times the nominal cracking moment strength is imposed to prevent brittle failures. This situation may occur where a beam is so lightly reinforced that the bending moment required to cause yielding of the reinforcement is less than the bending moment required to cause cracking.

3.2.4.2.3 Transverse reinforcement — Beams recognized in this section of the Code are often designed to resist only shear forces due to gravity loads. Flexural elements that are controlled by high seismic forces and lateral drift should be designed as ductile elements.

(a) Although some concerns have been raised regarding the difficulty in constructing beams containing a single bar stirrup, the Committee feels such spacing limitations within beams inhibits the construction of necessary lap lengths required for two-bar stirrups. Furthermore, the added volume of reinforcing steel as a result of lap splicing stirrups may prevent adequate consolidation of the grout.

(b) The requirement that shear reinforcement be hooked around the longitudinal reinforcement not only facilitates construction but also confines the longitudinal reinforcement and helps ensure the development of the shear reinforcement.

(c) A minimum area of transverse reinforcement is established to prevent brittle shear failures.

(d) Although different codes contain different spacing requirements for the placement of transverse reinforcement, the Committee has conservatively established this requirement.

(e) The reinforcement requirements of this section establish limitations on the spacing and placement of steel in order to increase member ductility.

3.2.4.2.4 Construction — Although beams can physically be constructed of partially grouted masonry, the lack of research supporting the performance of partially grouted beams combined with the increased probability of brittle failure dictates this requirement.
3.2.4.2.5 Dimensional limits
(a) This limitation is judgement-based. The minimum spacing of lateral support for masonry beams is imposed to ensure lateral stability of the element.
(b) Insufficient research has been conducted on beams of nominal depth less than 8 in. (203 mm).

3.2.4.3 Piers
3.2.4.3.1 Due to the less severe requirements imposed for the design of piers with respect to similar requirements for columns, the maximum axial force is arbitrarily limited to a relatively lower value.

3.2.4.3.2 Longitudinal reinforcement — These provisions are predominantly seismic related and are intended to provide the greatest ductility for the least expense. Pier elements not subject to in-plane stress reversals are not required to comply with this section.

3.2.4.3.3 Dimensional limits — Judgement-based dimensional limits are established for pier elements to distinguish their design from walls and to prevent local instability or buckling modes.

3.2.4.4 Columns
3.2.4.4.4 Dimensional limits — These limitations are judgement-based. They are intended to prevent local instability or buckling modes.

3.2.5 Wall design for out-of-plane loads
3.2.5.2 Maximum reinforcement — Refer to Commentary Section 3.2.3.5 for additional information.
3.2.5.3 Moment and deflection calculations — The provisions of this section are derived from test specimens that were modeled with pin-connections top and bottom, thus having simple support conditions. As a result, the maximum bending moment and deflection occurred at the mid-height of the specimens – hence, this section only includes design equations based on these observations. In actual design and construction, there may be varying support conditions, thus changing the curvature of the wall under lateral loading. Through proper calculation, using the principles of mechanics, the points of inflection can be determined and actual moments and deflection can be calculated under different support conditions. The designer should examine all moment and deflection conditions to locate the critical section using the assumptions outlined in Section 3.2.5.
3.2.5.4 Walls with factored axial stress of 0.05 $f'_m$ or less — The criterion to limit vertical load on a cross section was included since the test data used to establish the slender wall design method was based on typical roof loads. A limit of 0.05 $f'_m$ was selected as a point of demarcation, below which the loads are not considered to significantly influence the design.

The required moment due to lateral loads, eccentricity of axial load, and lateral deformations are assumed maximum at mid-height of the wall. In certain design conditions, such as large eccentricities acting simultaneously with small lateral loads, the design maximum moment may occur elsewhere. When this occurs, the designer should use the maximum moment at the critical section rather than the moment determined from Eq. (3-25).

The design formulas provide procedures for determining the nominal moment strength. These formulas take into account the effect of vertical loads increasing the capacity of the section.

3.2.5.5 Walls with factored axial stress greater than 0.05 $f'_m$ — The design of walls falling into this category follows the same procedures as previously outlined above, with the additional limitation of the aspect ratio of the wall.

3.2.5.6 Deflection design — Historically, the recommendation has been to limit the deflection under service load to 0.01h. However, this deflection control has also been viewed as neither a safety feature nor a safety requirement, but instead as an aesthetic and serviceability requirement considering intersecting walls and attachment to other materials. Hence, to address the concerns associated with safety, the deflection limitation of elements under service load was reduced to a value of 0.007h.

The Code limits the lateral deflection under service loads. The wall will rebound to its normal vertical conditions when the lateral load is removed because the stress in the reinforcement is within its elastic limit.

Eq. (3-30) is for mid-height deflection for an uncracked section. Eq. (3-31) is for mid-height deflection for a cracked section. Assume that the wall deflects as an uncracked section until the modulus of rupture is reached. In these formulas, the effective area is used to calculate the moment of inertia, $I_e$. After the modulus of rupture is reached, the wall cracks and the moment of inertia is reduced. Thus, the cracked moment of inertia is assumed for the full height of the wall in calculating the additional deflection. The cracked moment of inertia, $I_{cr}$, for a solid grouted or partially grouted cross section is usually the same as that for a hollow section since the compression stress block is generally within the thickness of the face shell.

These formulas are good approximations to the results achieved in the tests, assuming the wall is pin-connected top and bottom in a simple flexural condition, and the wall is subjected to a uniform distributed lateral load. If the wall is fixed at top or bottom or both, other formulas should be developed considering the support conditions at the top or bottom and considering the possible deflection or rotation of the foundation, roof, or floor diaphragm.

The section modulus in Eq. (3-32) is the effective moment of inertia divided by the distance from the neutral axis to the extreme fiber in tension. The effective moment of inertia would be based upon the minimum solid net areas of solid or partially grouted sections. The
cracking moment, \( M_{cr} \), is the calculated moment corresponding to first cracking.

### 3.3 — Unreinforced (plain) masonry

#### 3.3.1.2 Strength contribution from reinforcement

Although reinforcement may still be present in unreinforced masonry, it is not considered in resisting design loads.

#### 3.3.1.3 Design criteria

The design of unreinforced masonry requires that the structure performs elastically under anticipated design loads. The system response factors used in the design of unreinforced masonry assume an elastic response of the structure.

#### 3.3.3 Nominal axial strength of unreinforced (plain) masonry members

Refer to Commentary Section 3.2.4.1.1 for additional information.

### References


2. The following Technical Coordinating Committee for Masonry Research (TCCMaR) task reports not specifically cited in this Chapter provide the substantiating data for the strength design criteria presented.


CHAPTER 4
PRESTRESSED MASONRY

4.1 — General

4.1.1 Scope

Prestressing forces are used in masonry to reduce or eliminate tensile stresses due to externally applied loads by using controlled precompression. The precompression is generated by prestressing tendons, either bars, wires, or strands, that are contained in openings in the masonry which may be grouted. The prestressing tendons can be pre-tensioned (stressed against external abutments prior to placing the masonry), or post-tensioned (stressed against the masonry after it has been placed).

Most construction applications to date have involved post-tensioned, ungrouted masonry for its ease of construction and overall economy. Consequently, these code provisions primarily focus on post-tensioned masonry. Although not very common, pre-tensioning has been used to construct prefabricated masonry panels. A more detailed review of prestressed masonry systems and applications can be found elsewhere.

Throughout this Code and Specification, references to “reinforcement” apply to non-prestressed reinforcement. These references do not apply to prestressing tendons, except as explicitly noted in Chapter 4. All requirements for prestressing tendons will be stated with the use of the terms “prestressing tendon” or “tendon”.

Section 1.13.2.1 includes strength design provisions for masonry members subjected to the strength-level, seismic loading conditions stipulated in ASCE 7. These provisions may be applied to prestressed masonry members with one exception. The moment strength of prestressed masonry members with laterally restrained prestressing tendons should be computed in accordance with the strength design method in Section 4.5.3.3 rather than the strength design method in Section 2.1.3.3.2.

The provisions of Chapter 4 do not require a mandatory quantity of reinforcement or bonded prestressing tendons for prestressed masonry members. However, the reinforcement requirements of Chapter 1 and Section 2.1 for masonry columns are applicable to prestressed masonry members. In this case, bonded as well as unbonded but laterally restrained tendons may be considered to contribute to the minimum reinforcement requirement. Tendons contribute their force to the moment strength as compared with the moment strength provided by the minimum reinforcement in Section 2.1.6.4. In the case where the tendons remain in tension at moment strength conditions, they are not subjected to buckling, and hence, do not need lateral support by the corner of a lateral tie. For reinforcement requirements of Section 1.13 for Seismic Design Categories C, D, E, and F bonded prestressing tendons may be considered to contribute to the minimum reinforcement requirements by their cross-sectional area. Unbonded prestressing tendons should not be considered to contribute to these minimum reinforcement requirements.

4.2 — Design methods

The typical method for design of prestressed masonry is an allowable stress design procedure. For the design of prestressed masonry with laterally restrained prestressing tendons, a combination of allowable stress design and strength design is proposed. A moment strength check is included for members with laterally restrained prestressing tendons to ensure adequate strength and ductility of the member in flexure under strength level loading. For simplicity and wherever possible, the Code requirements for unreinforced masonry have been applied to prestressed masonry. The British code for prestressed masonry and extensive research of the properties of prestressed masonry have also been considered. Summaries of prestressed masonry research and proposed design criteria are available in the literature.

Often, the masonry member will be prestressed prior to 28 days after construction. The specified compressive strength of the masonry at the time of prestressing \( f'_{cm} \) is used to determine allowable prestressing levels. This strength will likely be a fraction of the 28-day specified compressive strength. Assessment of compressive strength at the time of prestress transfer should be by testing of masonry prisms or by a record of strength gain over time of masonry prisms constructed of similar masonry units, mortar, grout, and prestressing grout when subjected to similar curing conditions.

4.3 — Permissible stresses in prestressing tendons

Allowable, prestressing-tendon stresses are based on criteria established for prestressed concrete. Allowable, prestressing-tendon stresses are for jacking forces and for the state of stress in the prestressing tendon immediately after the prestressing has been applied, or transferred, to the masonry. When computing the prestressing-tendon stress immediately after transfer of prestress, consider all sources of short term prestress losses. These sources include such items as anchorage seating loss, elastic shortening of masonry, and friction losses.

4.4 — Effective prestress

The state of stress in a prestressed masonry member must be checked for all stages of loading. For each loading condition, the effective level of prestress should be used in the computation of stresses and member strength. Effective prestress is not a fixed quantity over time. Research on the loss and gain of prestress in prestressed masonry is extensive and includes testing of
time-dependent phenomena such as creep, shrinkage, moisture expansion, and prestressing-tendon stress relaxation.\textsuperscript{4.10 - 4.13}

Instantaneous deformation of masonry due to the application of prestress may be computed by the modulus of elasticity of masonry given in Section 1.8.2. Creep, shrinkage, and moisture expansion of masonry may be computed by the coefficients given in Section 1.8. Change in effective prestress due to elastic deformation, creep, shrinkage, and moisture expansion should be based on relative modulus of elasticity of masonry and prestressing steel.

The stressing operation and relative placement of prestressing tendons should be considered in calculating losses. Elastic shortening during post-tensioning can reduce the stress in adjacent tendons that have already been stressed. Consequently, elastic shortening of the wall should be calculated considering the incremental application of post-tensioning. That elastic shortening should then be used to estimate the total loss of prestress. Alternatively, post-tensioning tendons can be prestressed to compensate for the elastic shortening caused by the incremental stressing operation.

Prestressing steel that is stressed to a large fraction of its yield stress and held at a constant strain will relax, requiring less stress to maintain a constant strain. The phenomenon of stress relaxation is associated with plastic deformation and its magnitude increases with steel stress as a fraction of steel strength. ASTM A 416, A 421, and A 722.\textsuperscript{4.14, 4.15, 4.16} Prestressing steels are stabilized for low relaxation losses during production. Other steel types that do not have this stabilization treatment may exhibit considerably higher relaxation losses. Their relaxation losses must be carefully assessed by testing. The loss of effective prestress due to stress relaxation of the prestressing tendon is dependent upon the level of prestress, which changes with time-dependent phenomenon such as creep, shrinkage, and moisture expansion of the masonry. An appropriate formula for predicting prestress loss due to relaxation has been developed.\textsuperscript{4.11 - 4.13} Alternately, direct addition of the steel stress relaxation value provided by the manufacturer can be used to compute prestress losses and gains.

Friction losses are minimal or nonexistent for most post-tensioned masonry applications, because prestressing tendons are usually straight and contained in cavities. For anchorage losses, manufacturers' information should be used to compute prestress losses. Changes in prestress due to thermal fluctuations may be neglected if masonry is prestressed with high-strength prestressing steels. Loss of prestressing should be calculated for each design to determine effective prestress. Calculations should be based on the particular construction materials and methods as well as the climate and environmental conditions. Committee experience with post-tensioned wall designs from several other countries have indicated that prestress losses are expected to be in the following ranges:

(a) Initial loss after jacking – 1% to 3%
(b) Total losses after long-term service for concrete masonry – 30% to 35%
(c) Total losses after long-term service for clay masonry – 20% to 25%

The values in (b) and (c) include both the short-term and long-term losses expected for post-tensioning. The Committee believes these ranges provide reasonable estimates for typical wall applications unless calculations, experience, or construction techniques indicate different losses are expected.

4.5 — Axial compression and flexure

4.5.1 General

The requirements for prestressed masonry members subjected to axial compression and flexure are separated into those with laterally unrestrained prestressing tendons and those with laterally restrained prestressing tendons. This separation was necessary because the flexural behavior of a prestressed masonry member significantly depends upon the lateral restraint of the prestressing tendon. Lateral restraint of a prestressing tendon is typically provided by grouting the cell or void containing the tendon before or after transfer of prestressing force to the masonry. Alternatively, lateral restraint may be provided by building the masonry into contact with the tendon or the tendon's protective sheathing at periodic intervals along the length of the prestressing tendon.

Allowable compressive stresses for prestressed masonry address two distinct loading stages; stresses immediately after transfer of prestressing force to the masonry member and stresses after all prestress losses and gains have taken place. The magnitude of allowable axial compressive stress and bending compressive stress after all prestress losses and gains are consistent with those for unreinforced and reinforced masonry in Sections 2.2 and 2.3, respectively. Immediately after transfer of prestressing, allowable compressive stresses and applied axial load should be based upon $f_{cm}$ and may be increased by 20 percent. This means that the factors of safety at the time of the transfer of prestress may be lower than those after prestress losses and gains occur. The first reason for this is that the effective precompression stress at the time of transfer of prestressing almost certainly decreases over time and masonry compressive strength most likely increases over time. Second, loads at the time of transfer of prestressing, namely prestress force and dead loads, are known more precisely than loads throughout the remainder of service life.

Cracking of prestressed masonry with laterally restrained or laterally unrestrained prestressing tendons under permanent loads is to be avoided. The prestressing force and the dead weight of the member are permanent loads. Cracking under permanent loading conditions is not
desirable due to the potential for significant water penetration, which may precipitate corrosion of the prestressing tendons and accessories and damage to interior finishes. Masonry provides a significant flexural tensile resistance to cracking, as reflected by the allowable flexural tensile stress values stated in Section 2.2. Consequently, elimination of tensile stress under prestressing force and dead loads alone is a conservative measure, but one the committee deemed reasonable and reflective of current practice for prestressed masonry members.

4.5.2 Laterally unrestrained prestressing tendons

Since masonry members with laterally unrestrained prestressing tendons are equivalent to masonry members subjected to applied axial loads, the design approach for unreinforced masonry in Section 2.2 has been adopted for convenience and consistency. Buckling of masonry members under prestressing force must be avoided for members with laterally unrestrained prestressing tendons. The prestressing force, \( P_{ps} \), is to be added to the design axial load, \( P \), for all stress and load computations and in the computation of the eccentricity of the axial resultant, \( e \).

4.5.3 Laterally restrained prestressing tendon

Lateral restraint of a prestressing tendon is typically provided by grouting the cell or void containing the tendon before or after transfer of prestressing force to the masonry. Alternatively, lateral restraint may be provided by building the masonry into contact with the tendon or the tendon’s protective sheath at periodic intervals along the length of the prestressing tendon. In general, three intermediate contacts within a laterally-unsupported wall length or height can be considered to provide full lateral support of the tendon.

Prestressed masonry members with laterally restrained prestressing tendons require a modified design approach from the criteria in Section 2.2. If the prestressing tendon is laterally restrained, the member cannot buckle under its own prestressing force. Any tendency to buckle under prestressing force will induce a lateral deformation that is resisted by an equal and opposite restraining force provided by the prestressing tendon. However, such members are susceptible to buckling under axial loads other than prestressing and this loading condition must be checked.\(^{4.11}\) For this condition, with both concentrically and eccentrically prestressed masonry members, the prestressing force must be considered in the computation of the eccentricity of this axial resultant, \( e \), in Eq. (2-15) of the Code. The flexural stress induced by eccentric prestressing will cause an increase or decrease in the axial buckling load, depending upon the location and magnitude of the applied axial load relative to the prestressing force.

Computation of the moment strength of prestressed masonry members with laterally restrained prestressing tendons is similar to the method for prestressed concrete.\(^ {4.9}\) The equation for the unbonded prestressing tendon stress, \( f_{ps} \), at the moment strength condition (Eq. 4-2) is based on tests of prestressed masonry members. The simplification of taking the tendon stress at nominal moment strength equal to the yield stress can be more conservative for bars than for strands because the yield stress of a prestressing bar is a smaller percentage of the tendon's ultimate strength. The equation for the nominal moment strength, \( M_{n} \), is for the general case of a masonry member with concentrically applied axial load and concentric tendons and reinforcement. This is representative of most prestressed masonry applications to date. For other conditions, the designer should refer to first principles to determine the nominal moment strength of the member.

The depth of the equivalent compression zone must be determined with consideration of the cross section of the member, the tensile stress of tendons and reinforcement, and the factored design axial load, \( P_{u} \). \( P_{u} \) is an additive quantity in Code Eqs. (4-1) and (4-3). Prestressing adds to the resistance for ultimate strength evaluations and is used with a load factor of 1.0.

The ratio, \( a/d \), must be less than 0.425 to promote a ductile failure in flexure. This limitation will require significant yielding of the prestressing tendons prior to masonry compression failure. In such a situation, the nominal moment strength is determined by the strength of the prestressing tendon, which is the basis for a strength reduction factor equal to 0.8.

4.6 — Axial tension

The axial tensile stress of masonry in a prestressed masonry member is to be neglected, which is a conservative measure. This requirement is consistent with that of Section 2.3. If axial tension develops, for example due to wind uplift on the roof structure, the axial tension must be resisted by reinforcement or tendons or both.

4.7 — Shear

This section applies to both in-plane and out-of-plane shear.

The enhancement of shear resistance provided by prestressing is represented by a Coulomb friction expression, as is done for unreinforced masonry in Section 2.2. The allowable shear stress is based on a shear stress calculation that assumes a parabolic shear stress distribution. Also included are allowable shear stress equations to limit the principal tensile stress (Equation 4-4b) and the principal compressive stress (Equation 4-4c), respectively. Equation (4-4b) is derived from:

\[
F_v = \sqrt{\left(1.5\sqrt{f_{w}}\right)^2 + \left(1.5\sqrt{f_{c}}\right)N_v / A_u}
\]

(This equation is for inch-pound units only.)
Equation (4-4c) limits the principal compressive stress.

To avoid high shear stresses that could cause masonry crushing, a limitation on the principal compression stress is used. A safety factor of 4.0 has been applied, which is consistent with the safety factor for compression in Sections 2.2 and 2.3. For fully grouted masonry, the strength is nearly isotropic. Hence, at any load direction the principal compression stress may not exceed an allowable compression stress of 0.25$f'_{m}$. For ungrouted or partially grouted masonry, the strength may drop below $f'_{m}$ in certain load directions. This fact is recognized by limiting the principal compression stress to 0.15$f'_{m}$, again assuming a safety factor of 4.0. Failure of the masonry in shear by exceeding the principal compressive strength requires large levels of prestressing and axial loading of the member.

No shear strength enhancement due to arching action of the masonry is recognized in this Code for prestressed masonry members. The formation of compression struts and tension ties in prestressed masonry is possible, but this phenomenon has not been considered.

4.8 — Deflection

In accordance with Chapter 1, prestressed masonry member deflection should be computed based on uncracked section properties. Computation of member deflection must include the effect of time-dependent phenomenon such as creep and shrinkage of masonry and relaxation of prestressing tendons. There are no limits for the out-of-plane deflection of prestressed masonry walls. This is because appropriate out-of-plane deflection limits will be project specific. The designer should consider the potential for damage to interior finishes and limit deflections accordingly.

4.9 — Prestressing tendon anchorages, couplers, and end blocks

The provisions of this section of the Code are used to design the tendon anchorages, couplers, and end blocks to withstand the prestressing operation and effectively transfer prestress force to the masonry member without distress to the masonry or the prestressing accessories. Anchorages are designed for adequate pull-out strength from their foundations.

Because the actual stresses are quite complicated around post-tensioning anchorages, experimental data, or a refined analysis should be used whenever possible. Appropriate formulas from the references 4.18 should be used as a guide to size prestressing tendon anchorages when experimental data or more refined analysis are not available. Additional guidance on design and details for post-tensioning anchorage zones is given in the references 4.19.

4.10 — Protection of prestressing tendons and accessories

Corrosion protection of the prestressing tendon and accessories is required in masonry walls subject to a moist and corrosive environment. Methods of corrosion protection are addressed in the Specification. Masonry and grout cover is not considered adequate protection due to variable permeability and the sensitivity of prestressing tendons to corrosion. The methods of corrosion protection given in the Specification provide a minimum level of corrosion protection. The designer may wish to impose more substantial corrosion protection requirements, especially in highly corrosive environments.

4.11 — Development of bonded tendons

Consistent with design practice in prestressed concrete, development of post-tensioned tendons away from the anchorage does not need to be calculated.

References


4.9. *Building Code Requirements for Reinforced Concrete*, ACI 318-95, American Concrete Institute, Detroit, MI, 1995.
5.1 — General

Empirical rules and formulas for the design of masonry structures were developed by experience. These are part of the legacy of masonry’s long use, predating engineering analysis. Design is based on the condition that gravity loads are reasonably centered on the bearing walls and the effect of any steel reinforcement, if used, is neglected. The masonry should be laid in running bond. Specific limitations on building height, seismic, wind and horizontal loads exist. Buildings are of limited height. Members not participating in the lateral force-resisting system of a building may be empirically designed even though the lateral force-resisting system is designed under Chapter 2.

These procedures have been compiled through the years. The most recent of these documents is the basis for this chapter.

Empirical design is a procedure of sizing and proportioning masonry elements. It is not design analysis. This procedure is conservative for most masonry construction. Empirical design of masonry was developed for buildings of smaller scale, with more masonry interior walls and stiffer floor systems than built today. Thus, the limits imposed are valid.

Since empirically-designed masonry is based on the gross compressive strength of the units, there is no need to specify the compressive strength of masonry.

5.3 — Lateral stability

Lateral stability requirements are a key provision of empirical design. Obviously, shear walls must be in two directions to provide stability. Bearing walls can serve as shear walls. See Fig. 5.3-1 for cumulative length of shear walls. The height of an element refers to the shortest unsupported height in the plane of the wall such as the shorter of a window jamb on one side and a door jamb on the other.

Fig. 5.3-1 — Cumulative length of shear walls
Diaphragm Panel Length = Dimension perpendicular to the resisting shear wall
Diaphragm Panel Width = Dimension parallel to the resisting shear wall

For example:
For Shear Walls A and B, the diaphragm panel length to width ratio is $X_1/Y$

For Shear Walls D and E, the diaphragm panel length to width ratio is $Y/X_1$

Note: Shear walls should be placed on all four sides of the diaphragm panel or the resulting torsion should be accounted for.

Fig. 5.3-2 — Diaphragm panel length to width ratio determination for shear wall spacing

5.4 — Compressive stress requirements
These are average compressive stresses based on gross area using actual dimensions. The following conditions should be used as guidelines when concentrated loads are placed on masonry:

- For concentrated loads acting on the full wall thickness, the allowable stresses under the load may be increased by 25 percent.
- For concentrated loads acting on concentrically placed bearing plates greater than one-half but less than full area, the allowable stress under the bearing plate may be increased by 50 percent.

The course immediately under the point of bearing should be a solid unit or filled solid with mortar or grout.

5.5 — Lateral support
Lateral support requirements are included to limit the flexural tensile stress due to out-of-plane loads. Masonry headers resist shear stress and permit the entire cross-section to perform as a single element. This is not the case for non-composite walls connected with wall ties. For such non-composite walls, the use of the sum of the thicknesses of the wythes has been used successfully for a long time, and is a traditional approach that is acceptable within the limits imposed by Code Table 5.5.1.

5.6 — Thickness of masonry

5.6.1 Experience of the committee has shown that the present ANSI A 41.1-5 thickness ratios are not always conservative. These requirements represent the consensus of the committee for more conservative design.

5.6.3 Foundation walls
Empirical criteria for masonry foundation wall thickness related to the depth of unbalanced fill have been contained in building codes and federal government standards for many years. The use of Code Table 5.6.3.1, which lists the traditional allowable backfill depths, is limited by a number of requirements that were not specified in previous codes and standards. These restrictions are enumerated in Section 5.6.3.1. Further precautions are recommended to guard against allowing heavy earth-moving or other equipment near enough to the foundation wall to develop high earth pressures.
Experience with local conditions should be used to modify the values in Table 5.6.3.1 when appropriate.

5.6.4 Foundation piers

Use of empirically-designed foundation piers has been common practice in many areas of the country for many years. ANSI A 41.1 provisions for empirically-designed piers (Section 5.3) included a requirement for a maximum h/t ratio of 4. The minimum height-to-thickness ratio of greater than 4 for columns is required to clearly differentiate a column from a pier.

5.7 Bond

Fig. 5.7-1 depicts the requirements listed. Wall ties with drips are not permitted because of their reduced load capacity.

Fig. 5.7-1 — Cross section of wall elevations
5.8 — Anchorage
The requirements of Sections 5.8.2.2 through 5.8.2.5 are less stringent than those of Section 1.9.4.2.5.

5.9 — Miscellaneous requirements
5.9.4 Corbelling
The provision for corbelling up to one-half of the wall or unit thickness is valid only if the opposite side of the wall remains in its same plane. See Fig. 5.9-1 for maximum unit projection.

References


Fig. 5.9-1 — Limitations on corbelling

Limitations on Corbelling:
\[ p \leq \frac{h}{2} \]
\[ p \leq \frac{d}{3} \]

Where:
\( p \) = Allowable Total Horizontal Projection of Corbelling,
\( p \) = Allowable Projection of One Unit,
\( t \) = Nominal Wall Thickness or One-half of the Wythe Thickness for Hollow Walls (actual thickness plus the thickness of one mortar joint),
\( h \) = Nominal Unit Height (actual height plus the thickness of one mortar joint),
\( d \) = Nominal Unit Bed Depth (actual bed depth plus the thickness of one mortar joint).
CHAPTER 6
VENeer

6.1 — General

6.1.1 Scope

Adhered and anchored veneer definitions found in Section 1.6 are straightforward adaptations of existing definitions. See Figs. 6.1-1 and 6.1-2 for typical examples of anchored and adhered veneer, respectively.

The traditional definition of veneer as an element without resistance to imposed load is adopted. The definition given is a variation of that found in model building codes. Modifications have been made to the definitions to clearly state how the veneer is handled in design.

The design of the backing should be in compliance with the appropriate standard for that material. Suggested standards are:

- concrete ......ACI 318, Building Code Requirements for Reinforced Concrete
- masonry ......Chapters 1 through 5 of this Code
- steel .............Design for Cold-formed Steel Structural Members
- wood ...........National Design Specification for Wood Construction

6.1.1.1 Since there is no consideration of stress in the veneer, there is no need to specify the compressive strength of masonry.

6.1.1.3 The Specification was written for construction of masonry subjected to design stresses in accordance with the other chapters of this Code. Masonry veneer, as defined by this Code, is not subject to those design provisions. The Specification articles that are excluded cover materials and requirements that are not applicable to veneer construction or are items covered by specific requirements in this Chapter and are put here to be inclusive.

6.1.2 Design of anchored veneer

Implicit within these requirements is the knowledge that the veneer transfers out-of-plane loads through the veneer anchors to the backing. The backing accepts and resists all anchor loads and is designed to resist all out-of-plane loads.

When utilizing anchored masonry veneer, the designer should consider the following conditions and assumptions:

a) The veneer may crack in flexure under service load.

b) Deflection of the backing should be limited to control crack width in the veneer and to provide veneer stability.

c) Connections of the anchor to the veneer and to the backing should be sufficient to transfer applied loads.

d) Differential movement should be considered in the design, detailing and construction.

e) Water will penetrate the veneer and the wall system should be designed, detailed and constructed to prevent water penetration into the building.

f) Requirements for corrosion protection and fire resistance must be included.

If the backing is masonry and the exterior masonry wythe is not considered to add to the out-of-plane load resisting performance of the wall, the exterior wythe is masonry veneer. However, if the exterior wythe is considered to add to the load-resisting performance of the wall, the wall is properly termed a multiwythe, non-composite wall rather than a veneer wall. Such walls are designed under Chapters 2 and 4 of this Code.

Manufacturers of steel studs and sheathing materials have published literature on the design of steel stud backing for anchored masonry veneer. Some recommendations have included composite action between the stud and the sheathing and load carrying participation by the veneer. The Metal Lath/Steel Framing Association has promoted a deflection limit of stud span length divided by 360. The Brick Industry Association has held that an appropriate deflection limit should be in the range of stud span length divided by 600 to 720. The deflection is computed assuming that all of the load is resisted by the studs. Neither set of assumptions will necessarily ensure that the veneer remains uncracked at service load. In fact, the probability of cracking may be high. However, post-cracking performance is satisfactory if the wall is properly designed, constructed and maintained with appropriate materials.

Plane frame computer programs are available for the rational structural design of anchored masonry veneer.

A deflection limit of stud span length divided by 200 times the specified veneer thickness provides a maximum uniform crack width for various heights and various veneer thicknesses. Deflection limits do not reflect the actual distribution of load. They are simply a means of obtaining a minimum backing stiffness. The National Concrete Masonry Association provides a design methodology by which the stiffness properties of the masonry veneer and its backing are proportioned to achieve compatibility.

Masonry veneer with wood frame backing has been used successfully on one- and two-family residential construction for many years. Most of these applications are installed without a deflection analysis.
Fig. 6.1-1 — Anchored veneer

Fig. 6.1-2 — Adhered veneer
6.1.3 Design of adhered veneer

Adhered veneer differs from anchored veneer in its means of attachment. The designer should consider conditions and assumptions given in Code Section 6.3.1 when designing adhered veneer.

6.1.4 Dimension stone

Dimension stone veneer should be covered as a Special System of Construction, under Code Section 1.3.

6.1.5 General design requirements

Water penetration through the exterior veneer is expected. The wall system must be designed and constructed to prevent water from entering the building.

The requirements given here and the minimum air space dimensions of Sections 6.2.2.6.3, 6.2.2.7.4, and 6.2.2.8.2 are those required for a drainage wall system. Proper drainage requires weep holes and a clear air space. It may be difficult to keep a 1 in. (25 mm) air space free from mortar bridging. Other options are to provide a wider air space, a vented air space, or to use the rain screen principle.

6.2 — Anchored veneer

6.2.1 Alternative design of anchored masonry veneer

There are no rational design provisions for anchored veneer in any code or standard. The intent of Section 6.2.1 is to permit the designer to use alternative means of supporting and anchoring masonry veneer. See Commentary Section 6.1.1 for conditions and assumptions to consider. The designer may choose to not consider stresses in the veneer or may limit them to a selected value such as the allowable stresses of Section 2.2, the anticipated cracking stress, or some other limiting condition. The rational analysis used to distribute the loads must be consistent with the assumptions made. See Commentary Section 6.2.2.5 for information on anchors.

The designer should provide support of the veneer; control deflection of the backing; consider anchor loads, stiffness, strength and corrosion; water penetration; and air and vapor transmission.

6.2.2 Prescriptive requirements for anchored masonry veneer

The provisions are based on the successful performance of anchored masonry veneer. These have been collected from a variety of sources and reflect current industry practices. Changes result from logical conclusions based on engineering consideration of the backing, anchor, and veneer performance.

6.2.2.3 Vertical support of anchored masonry veneer — These requirements are based on current industry practice and current model building codes.

Support does not need to occur at the floor level; it can occur at a window head or other convenient location.

The full provisions for preservative-treated wood foundations are found in the National Forest Products Association Technical Report 7.6.9

There are no restrictions on the height limit of veneer backed by masonry or concrete, nor are there any requirements that the veneer weight be carried by intermediate supports. The designer should consider the effects of differential movement on the anchors and connection of the veneer to other building components.

Support of anchored veneer on wood is permitted in previous model building codes. The vertical movement joint between the veneer on different supports reduces the possibility of cracking due to differential settlement. The height limit of 12 ft (3.7 m) was considered to be the maximum single story height and is considered to be a reasonable fire safety risk.

6.2.2.5 Anchor requirements — It could be argued that the device between the veneer and its backing is not an anchor as defined in the Code. That device is often referred to as a tie. However, the term anchor is used because of the widespread use of anchored veneer in model building codes and industry publications, and the desire to differentiate from tie as used in other chapters.

U.S. industry practice has been combined with the requirements of the Canadian Standards Association6.10 to produce the requirements given. Each anchor type has physical requirements that must be met. Minimum embedment requirements have been set for each of the anchor types to ensure load resistance against push-through or pull-out of the mortar joint. Maximum air space dimensions are set in Sections 6.2.2.6 through 6.2.2.8.

There are no performance requirements for veneer anchors in previous codes. Indeed, there are none in the industry. Tests on anchors have been reported6.4, 6.11. Many anchor manufacturers have strength and stiffness data for their proprietary anchors.

Veneer anchors typically allow for movement in the plane of the wall but resist movement perpendicular to the veneer. The mechanical play in adjustable anchors and the stiffness of the anchor will influence load transfer between the veneer and the backing. Stiff anchors with minimal mechanical play provide more uniform transfer of load, increase the stress in the veneer, and reduce veneer deflection.

The anchors listed in 6.2.2.5.6.1 are thought to have lower strength or stiffness than the more rigid plate-type anchors. Thus fewer plate-type anchors are required. These provisions may result in an increase in the number of anchors required when compared to the editions of the BOCA and SBCCI model building codes published in
The number of anchors decreases in low seismic zones from the requirements in the UBC. Anchor spacing is independent of backing type.

Anchor frequency should be calculated independently for the wall surface in each plane. That is, horizontal spacing of veneer anchors should not be continued from one plane of the veneer to another.

6.2.2.6 Masonry veneer anchored to wood backing — These requirements are similar to those used by industry and found in model building codes for years. The limitation on fastening corrugated anchors at a maximum distance from the bend is new. It is added to achieve better performance. The maximum distances between the veneer and the sheathing or wood stud is provided in order to obtain minimum compression capacity of anchors.

6.2.2.7 Masonry veneer anchored to steel backing — Most of these requirements are new, but they generally follow recommendations in current use. The minimum base metal thickness is given to provide sufficient pull-out resistance of screws.

6.2.2.8 Masonry veneer anchored to masonry or concrete backing — These requirements are similar to those used by industry and have been found in model building codes for many years.

6.2.2.9 Veneer laid in other than running bond — Masonry laid in other than running bond has similar requirements in Section 1.11. The area of steel required in Section 6.2.2.9 is equivalent to that in Section 1.11 for a nominal 4 in. (102 mm) wythe.

6.2.2.10 Requirements in seismic areas — These requirements provide several cumulative effects to improve veneer performance under seismic load. Many of them are based on similar requirements found in Chapter 30 of the Uniform Building Code. The isolation from the structure reduces accidental loading and permits larger building deflections to occur without veneer damage. Support at each floor articulates the veneer and reduces the size of potentially damaged areas. An increased number of anchors increases veneer stability and reduces the possibility of falling debris. Joint reinforcement provides ductility and post-cracking strength. Added expansion joints further articulate the veneer, permit greater building deflection without veneer damage and limit stress development in the veneer.

6.3 Adhered veneer

6.3.1 Alternative design of adhered masonry veneer

There are no rational design provisions for adhered veneer in any code or standard. The intent of Section 6.3.1 is to permit the designer to use alternative unit thicknesses and areas for adhered veneer.

The designer should provide for adhesion of the units, control curvature of the backing, and consider freeze-thaw cycling, water penetration, and air and vapor transmission. The Tile Council of America limits the deflection of the backing supporting ceramic tiles to span length divided by 360.6.16

6.3.2 Prescriptive requirements for adhered masonry veneer

Similar requirements for adhered veneer have been in the Uniform Building Code since 1967. The construction requirements for adhered veneer in the Specification have performed successfully.

6.3.2.1 Unit sizes — The dimension, area, and weight limits are imposed to reduce the difficulties of handling and installing large units and to assure good bond.

6.3.2.2 Wall area limitations — Selecting proper location for movement joints involves many variables. These include: changes in moisture content, inherent movement of materials, temperature exposure, temperature differentials, strength of units, and stiffness of the backing.

6.3.2.3 Backing — These surfaces have demonstrated the ability to provide the necessary adhesion when using the construction method described in the Specification. Model building codes contain provisions for metal lath and Portland cement plaster. For masonry or concrete backing, it may be desirable to apply metal lath and plaster. Also, refer to ACI 524R, "Guide to Portland Cement Plastering"6.18 for metal lath, accessories, and their installation. These publications also contain recommendations for control of cracking.

6.3.2.4 The required shear strength of 50 psi (345 kPa) is an empirical value based on judgment derived from historical use of adhered veneer systems similar to those permitted by Article 3.3 C of ACI 530.1/ASCE 6/TMS 602. This value is easily obtained with workmanship complying with the Specification. It is anticipated that the 50 psi (345 kPa) will account for differential shear stress between the veneer and its backing in adhered veneer systems permitted by this code and Specification.

The test method is used to verify shear strength of adhered veneer systems that do not comply with the construction requirements of the Specification or as a quality assurance test for systems that do comply.
References

6.1. Building Code Requirements for Reinforced Concrete, ACI 318-95, American Concrete Institute, Detroit, MI, 1995.


7.1 — General

7.1.1 Scope

Glass unit masonry is used as nonload-bearing elements in interior and exterior walls, and in window openings. Code provisions are empirical, based on previous codes, successful performance, and manufacturers' recommendations.

7.1.1.1 Since there is no consideration of stress in glass unit masonry, there is no need to specify the compressive strength of masonry.

7.2 — Panel size

The Code limitations on panel size are based on structural and performance considerations. Height limits are more restrictive than length limits based on historical requirements rather than actual field experience or engineering principles. Fire resistance rating tests of assemblies may also establish limitations on panel size. Contact glass block manufacturers for technical data on the fire resistance ratings of panels, or refer to the latest issue of UL Building Materials Directory and the local building code.

7.2.1 Exterior standard-unit panels

The wind load resistance curve (Fig. 7.2-1) is representative of the ultimate load limits for a variety of panel conditions. The 144 ft² (13.37 m²) area limit is based on a safety factor of 2.7 when the design wind pressure is 20 psf (958 Pa).

7.2.2 Exterior thin-unit panels

There is no historical data for developing a curve for thin units. The Committee recommends limiting the exterior use of thin units to areas where the design wind pressure does not exceed 20 psf (958 Pa).

7.3 — Support

7.3.3 Lateral

The Code requires glass unit masonry panels to be laterally supported by panel anchors or channel-type restraints. See Figs. 7.3-1 and 7.3-2 for panel anchor construction and channel-type restraint construction, respectively. Glass unit masonry panels may be laterally supported by either construction type or by a combination of construction types. The channel-type restraint construction can be made of any channel-shaped concrete, masonry, metal, or wood elements so long as they provide the required lateral support.

Example of how to use wind-load resistance curve: If using a design wind pressure of 20 psf (958 Pa), multiply by a safety factor of 2.7 and locate 54 psf (2586 Pa) wind pressure (on vertical axis), read across to curve and read corresponding 144 ft² (13.37 m²) maximum area per panel (on horizontal axis).

**Fig. 7.2-1 — Glass masonry ultimate wind load resistance**
**Fig. 7.3-1** — Panel anchor construction

**Fig. 7.3-2** — Channel-type restraint construction
7.5 — Base surface treatment

Current industry practice and recommendations by glass block manufacturers state that all surfaces on which glass unit masonry is placed be coated with an asphalt emulsion.\textsuperscript{7.2, 7.3} The asphalt emulsion provides a slip plane at the panel base. This is in addition to the expansion provisions at head and jamb locations. The asphalt emulsion also waterproofs porous panel bases.

Glass unit masonry panels subjected to structural investigation tests by the National Concrete Masonry Association\textsuperscript{7.5} to confirm the validity and use of the Glass Unit Masonry Design Wind Load Resistance chart (Fig. 7.2-1) of the Code, were constructed on bases coated with asphalt emulsion. Asphalt emulsion on glass unit masonry panel bases is needed to be consistent with these tests.

References


7.5. Structural Investigation of Pittsburgh Corning Glass Block Masonry, National Concrete Masonry Association Research and Development Laboratory, Herndon, VA, August 1992.