7. Masonry (Systematic Rehabilitation)

7.1 Scope

This chapter describes engineering procedures for estimating seismic performance of vertical lateral-force-resisting masonry elements. The methods are applicable for masonry wall and infill panels that are either existing or rehabilitated elements of a building system, or new elements that are added to an existing building system.

This chapter presents the information needed for systematic rehabilitation of masonry buildings as depicted in Step 3 of the Process Flow chart shown in Figure 1-1. A brief historical perspective is given in Section 7.2, with an expanded version in *Commentary* Section C7.2. Masonry material properties for new and existing construction are discussed in Section 7.3.

Attributes of masonry walls and masonry infills are given in Sections 7.4 and 7.5, respectively. Masonry components are classified by their behavior; unreinforced components precede reinforced components, and in-plane action is separated from out-of-plane action. For each component type, the information needed to model stiffness is presented first, followed by recommended strength and deformation acceptance criteria for various performance levels. These attributes are presented in a format for direct use with the Linear and Nonlinear Static Procedures prescribed in Chapter 3. Guidelines for anchorage to masonry walls and masonry foundation elements are given in Sections 7.6 and 7.7, respectively. Section 7.8 provides definitions for terms used in this chapter, and Section 7.9 lists the symbols used in Chapter 7 equations. Applicable reference standards are listed in Section 7.10.

Portions of a masonry building that are not subject to systematic rehabilitation provisions of this chapter—such as parapets, cladding, or partition walls—shall be considered with the Simplified Rehabilitation options of Chapter 10 or with the provisions for nonstructural components addressed in Chapter 11.

The provisions of this chapter are intended for solid or hollow clay-unit masonry, solid or hollow concrete-unit masonry, and hollow clay tile. Stone or glass block masonry is not covered in this chapter.

The properties and behavior of steel, concrete, and timber floor or roof diaphragms are addressed in Chapters 5, 6, and 8, respectively. Connections to masonry walls are addressed in Section 7.6 for cases where behavior of the connection is dependent on properties of the masonry. Attributes for masonry foundation elements are briefly described in Section 7.7.

Unreinforced masonry buildings with flexible floor diaphragms may be evaluated by using the procedures given in Appendix C of FEMA 178 (BSSC, 1992) if the simplified rehabilitation approach of Chapter 10 is followed.

7.2 Historical Perspective

Construction of existing masonry buildings in the United States dates back to the 1500s in the southeastern and southwestern parts of the country, to the 1770s in the central and eastern parts, and to the 1850s in the western half of the nation. The stock of existing masonry buildings in the United States largely comprises structures constructed in the last 150 years. Since the types of units, mortars, and construction methods have changed over this course of time, knowing the vintage of a masonry building may be useful in identifying the characteristics of the construction. Although structural properties cannot be inferred solely from age, some background on typical materials and methods for a given period can help to improve engineering judgment, and provide some direction in the assessment of an existing building.

As indicated in Chapter 1, great care should be exercised in selecting the appropriate rehabilitation approaches and techniques for application to historic buildings in order to preserve their unique characteristics.

Section C7.2 of the *Commentary* provides an extensive historical perspective on various masonry materials and construction practices.

7.3 Material Properties and Condition Assessment

7.3.1 General

The methods specified in Section 7.3.2 for determination of mechanical properties of existing masonry construction shall be used as the basis for stiffness and strength attributes of masonry walls and infill panels, along with the methods described in Sections 7.4 and 7.5. Properties of new masonry components that are added to an existing structural system shall be based on values given in BSSC (1995).

Minimum requirements for determining in situ masonry compressive, tensile, and shear strength, as well as elastic and shear moduli, are provided in Section 7.3.2. Recommended procedures for measurement of each material property are described in corresponding sections of the *Commentary*. Data from testing of in-place materials shall be expressed in terms of mean values for determination of expected component strengths, Q_{CE}, and lower bound strengths, Q_{CL}, with the linear or nonlinear procedures described in Chapter 3.

In lieu of in situ testing, default values of material strength and modulus, as given in Section 7.3.2, shall be assigned to masonry components in good, fair, and poor condition. Default values represent typical lower bound estimates of strength or stiffness for all masonry nationwide, and thus should not be construed as expected values for a specific structure. As specified in Section 7.3.4, certain in situ tests are needed to attain the comprehensive level of knowledge (a κ value of 1.00) needed in order to use the nonlinear procedures of Chapter 3. Thus, unless noted otherwise, general use of the default values without in situ testing is limited to the linear procedures of Chapter 3.

Procedures for defining masonry structural systems, and assessing masonry condition, shall be conducted in accordance with provisions stated in Section 7.3.3. Requirements for either a minimum or a comprehensive level of evaluation, as generally stated in Section 2.7, are further refined for masonry components in Section 7.3.4.

7.3.2 Properties of In-Place Materials

7.3.2.1 Masonry Compressive Strength

Expected masonry compressive strength, f_{me}, shall be measured using one of the following three methods.

1. Test prisms shall be extracted from an existing wall and tested per Section 1.4.B.3 of the Masonry Standards Joint Committee's *Building Code Requirements for Masonry Structures* (MSJC, 1995a).

2. Prisms shall be fabricated from actual extracted masonry units, and a surrogate mortar designed on the basis of a chemical analysis of actual mortar samples. The test prisms shall be tested per Section 1.4.B.3 of the *Specification for Masonry Structures* (MSJC, 1995b).

3. Two flat jacks shall be inserted into slots cut into mortar bed joints and pressurized until peak stress is reached.

For each of the three methods, the expected compressive strength shall be based on the net mortared area.

If the masonry unit strength and the mortar type are known, f_{me} values may be taken from Tables 1 and 2 of MSJC (1995a) for clay or concrete masonry constructed after 1960. The f_{me} value shall be obtained by multiplying the table values by a factor that represents both the ratio of expected to lower bound strength and the height-to-thickness ratio of the prism (see *Commentary* Section C7.3.2.1).

In lieu of material tests, default values for masonry prism compressive strength shall be taken to not exceed 900 psi for masonry in good condition, 600 psi for masonry in fair condition, and 300 psi for masonry in poor condition.

7.3.2.2 Masonry Elastic Modulus in Compression

Expected values of elastic modulus for masonry in compression, E_{me}, shall be measured using one of the following two methods:

1. Test prisms shall be extracted from an existing wall, transported to a laboratory, and tested in

compression. Stresses and deformations shall be measured to infer modulus values.

2. Two flat jacks shall be inserted into slots cut into mortar bed joints, and pressurized up to nominally one half of the expected masonry compressive strength. Deformations between the two flat jacks shall be measured to infer compressive strain, and thus elastic modulus.

In lieu of prism tests, values for the modulus of elasticity of masonry in compression shall be taken as 550 times the expected masonry compressive strength, f_{me}.

7.3.2.3 Masonry Flexural Tensile Strength

Expected flexural tensile strength, f_{te}, for out-of-plane bending shall be measured using one of the following three methods:

1. Test samples shall be extracted from an existing wall, and subjected to minor-axis bending using the bond-wrench method.

2. Test samples shall be tested in situ using the bond-wrench method.

3. Sample wall panels shall be extracted and subjected to minor-axis bending in accordance with ASTM E 518.

In lieu of material tests, default values of masonry flexural tensile strength for walls or infill panels loaded normal to their plane shall be taken to not exceed 20 psi for masonry in good condition, 10 psi for masonry in fair condition, and zero psi for masonry in poor condition. For masonry constructed after 1960 with cement-based mortars, default values of flexural tensile strength can be based on values from Table 8.3.10.5.1 of BSSC (1995).

Flexural tensile strength for unreinforced masonry (URM) walls subjected to in-plane lateral forces shall be assumed to be equal to that for out-of-plane bending, unless testing is done to define expected tensile strength.

7.3.2.4 Masonry Shear Strength

For URM components, expected masonry shear strength, v_{me}, shall be measured using the in-place shear test. Expected shear strength shall be determined in accordance with Equation 7-1.

$$v_{me} = \frac{0.75\left(0.75 v_{te} + \frac{P_{CE}}{A_n}\right)}{1.5} \qquad (7\text{-}1)$$

where

P_{CE} = Expected gravity compressive force applied to a wall or pier component stress considering load combinations given in Equations 3-14, 3-15, and 3-16

A_n = Area of net mortared/grouted section, in.^{2}

v_{te} = Average bed-joint shear strength, psi

The 0.75 factor on the v_{te} term may be waived for single wythe masonry, or if the collar joint is known to be absent or in very poor condition.

Values for the mortar shear strength, v_{te}, shall not exceed 100 psi for the determination of v_{me} in Equation 7-1.

Average bed-joint shear strength, v_{te}, shall be determined from individual shear strength test values, v_{to}, in accordance with Equation 7-2.

$$v_{to} = \frac{V_{test}}{A_b} - p_{D+L} \qquad (7\text{-}2)$$

where V_{test} is the load at first movement of a masonry unit, A_b is the net mortared area of the bed joints above and below the test brick, and p_{D+L} is the estimated gravity stress at the test location.

In lieu of material tests, default values of shear strength of URM components shall be taken to not exceed 27 psi for running bond masonry in good condition, 20 psi for running bond masonry in fair condition, and 13 psi for running bond masonry in poor condition. These values shall also be used for masonry in other than running bond if fully grouted. For masonry in other than running bond and partially grouted or ungrouted, shear strength shall be reduced by 60% of these values. For masonry constructed after 1960 with cement-based mortars, default values of shear strength can be based on values in BSSC (1995) for unreinforced masonry.

The in-place shear test shall not be used to estimate shear strength of reinforced masonry (RM) components. The expected shear strength of RM components shall be in accordance with Section 7.4.4.2A.

7.3.2.5 Masonry Shear Modulus

The expected shear modulus of uncracked, unreinforced, or reinforced masonry, G_{me}, shall be estimated as 0.4 times the elastic modulus in compression. After cracking, the shear modulus shall be taken as a fraction of this value based on the amount of bed joint sliding or the opening of diagonal tension cracks.

7.3.2.6 Strength and Modulus of Reinforcing Steel

The expected yield strength of reinforcing bars, f_{ye}, shall be based on mill test data, or tension tests of actual reinforcing bars taken from the subject building. Tension tests shall be done in accordance with ASTM A 615.

In lieu of tension tests of reinforcing bars, default values of yield stress shall be determined per Section 6.3.2.5. These values shall also be considered as lower bound values, f_y, to be used to estimate lower bound strengths, Q_{CL}.

The expected modulus of elasticity of steel reinforcement, E_{se}, shall be assumed to be 29,000,000 psi.

7.3.2.7 Location and Minimum Number of Tests

The number and location of material tests shall be selected to provide sufficient information to adequately define the existing condition of materials in the building. Test locations shall be identified in those masonry components that are determined to be critical to the primary path of lateral-force resistance.

A visual inspection of masonry condition shall be done in conjunction with any in situ material tests to assess uniformity of construction quality. For masonry with consistent quality, the minimum number of tests for each masonry type, and for each three floors of construction or 3000 square feet of wall surface, shall be three, if original construction records are available that specify material properties, or six, if original construction records are not available. At least two tests should be done per wall, or line of wall elements providing a common resistance to lateral forces. A minimum of eight tests should be done per building.

Tests should be taken at locations representative of the material conditions throughout the entire building, taking into account variations in workmanship at different story levels, variations in weathering of the exterior surfaces, and variations in the condition of the interior surfaces due to deterioration caused by leaks and condensation of water and/or the deleterious effects of other substances contained within the building.

For masonry with perceived inconsistent quality, additional tests shall be done as needed to estimate material strengths in regions where properties are suspected to differ. Nondestructive condition assessment tests per Section 7.3.3.2 may be used to quantify variations in material strengths.

An increased sample size may be adopted to improve the confidence level. The relation between sample size and confidence shall be as defined in ASTM E 22.

If the coefficient of variation in test measurements exceeds 25%, additional tests shall be done. If the variation does not reduce below this limit, use of the test data shall be limited to the Linear Static Procedures of Chapter 3.

If mean values from in situ material tests are less than the default values prescribed in Section 7.3.2, additional tests shall be done. If the mean continues to be less than the default values, the measured values shall be used, and shall be used only with the Linear Static Procedures of Chapter 3.

7.3.3 Condition Assessment

7.3.3.1 Visual Examination

The size and location of all masonry shear and bearing walls shall be determined. The orientation and placement of the walls shall be noted. Overall dimensions of masonry components shall be measured, or determined from plans, including wall heights, lengths, and thicknesses. Locations and sizes of window and door openings shall be measured, or determined from plans. The distribution of gravity loads to bearing walls should be estimated.

The wall type shall be identified as reinforced or unreinforced, composite or noncomposite, and/or grouted, partially grouted, or ungrouted. For RM construction, the size and spacing of horizontal and vertical reinforcement should be estimated. For multiwythe construction, the number of wythes should be noted, as well as the distance between wythes (the thickness of the collar joint or cavity), and the placement of interwythe ties. The condition and attachment of veneer wythes should be noted. For grouted construction, the quality of grout placement should be assessed. For partially grouted walls, the locations of grout placement should be identified.

The type and condition of the mortar and mortar joints shall be determined. Mortar shall be examined for weathering, erosion, and hardness, and to identify the condition of any repointing, including cracks, internal voids, weak components, and/or deteriorated or eroded mortar. Horizontal cracks in bed joints, vertical cracks in head joints and masonry units, and diagonal cracks near openings shall be noted.

The examination shall identify vertical components that are not straight. Bulging or undulations in walls shall be observed, as well as separation of exterior wythes, out-of-plumb walls, and leaning parapets or chimneys.

Connections between masonry walls, and between masonry walls and floors or roofs, shall be examined to identify details and condition. If construction drawings are available, a minimum of three connections shall be inspected for each general connection type (e.g., floor-to-wall, wall-to-wall). If no deviations from the drawings are found, the sample may be considered representative. If drawings are unavailable, or significant deviations are noted between the drawings and constructed work, then a random sample of connections shall be inspected until a representative pattern of connections can be identified.

7.3.3.2 Nondestructive Tests

Nondestructive tests may be used to supplement the visual observations required in Section 7.3.3.1. One, or a combination, of the following nondestructive tests, shall be done to meet the requirements of a comprehensive evaluation as stated in Section 7.3.4:

- ultrasonic pulse velocity
- mechanical pulse velocity
- impact echo
- radiography

The location and number of nondestructive tests shall be in accordance with the requirements of Section 7.3.2.7. Descriptive information concerning these test procedures is provided in the *Commentary*, Section C7.3.3.2.

7.3.3.3 Supplemental Tests

Ancillary tests are recommended, but not required, to enhance the level of confidence in masonry material properties, or to assess condition. These are described in the Commentary to this section.

7.3.4 Knowledge (κ) Factor

In addition to those characteristics specified in Section 2.7.2, a knowledge factor, κ, equal to 0.75, representing a minimum level of knowledge of the structural system, shall be used if a visual examination of masonry structural components is done per the requirements of Section 7.3.3.1. A knowledge factor, κ, equal to 1.00, shall be used only with a comprehensive level of knowledge of the structural system (as defined in Section 2.7.2).

7.4 Engineering Properties of Masonry Walls

This section provides basic engineering information for assessing attributes of structural walls, and includes stiffness assumptions, strength acceptance criteria, and deformation acceptance criteria for the Immediate Occupancy, Life Safety, and Collapse Prevention Performance Levels. Engineering properties given for masonry walls shall be used with the analytical methods prescribed in Chapter 3, unless otherwise noted.

Masonry walls shall be categorized as primary or secondary elements. Walls that are considered to be part of the lateral-force system, and may or may not support gravity loads, shall be primary elements. Walls that are not considered as part of the lateral-force-resisting system, but must remain stable while supporting gravity loads during seismic excitation, shall be secondary elements.

7.4.1 Types of Masonry Walls

The procedures set forth in this section are applicable to building systems comprising any combination of existing masonry walls, masonry walls enhanced for seismic rehabilitation, and new walls added to an existing building for seismic rehabilitation. In addition, any of these three categories of masonry elements can be used in combination with existing, rehabilitated, or new lateral-force-resisting elements of other materials such as steel, concrete, or timber.

When analyzing a system comprising existing masonry walls, rehabilitated masonry walls, and/or new masonry walls, expected values of strength and stiffness shall be used.

7.4.1.1 Existing Masonry Walls

Existing masonry walls considered in Section 7.4 shall include all structural walls of a building system that are in place prior to seismic rehabilitation.

Wall types shall include unreinforced or reinforced; ungrouted, partially grouted, or fully grouted; and composite or noncomposite. Existing walls subjected to lateral forces applied parallel with their plane shall be considered separately from walls subjected to forces applied normal to their plane, as described in Sections 7.4.2 through 7.4.5.

Material properties for existing walls shall be established per Section 7.3.2. Prior to rehabilitation, masonry structural walls shall be assessed for condition per the procedures set forth in Sections 7.3.3.1, 7.3.3.2, or 7.3.3.3. Existing masonry walls shall be assumed to behave in the same manner as new masonry walls, provided that the condition assessment demonstrates equivalent quality of construction.

7.4.1.2 New Masonry Walls

New masonry walls shall include all new elements added to an existing lateral-force-resisting system. Wall types shall include unreinforced or reinforced; ungrouted, partially grouted, or fully grouted; and composite or noncomposite. Design of newly constructed walls shall follow the requirements set forth in BSSC (1995).

When analyzing a system of new and existing walls, expected values of strength and stiffness shall be used for the newly constructed walls. Any capacity reduction factors given in BSSC (1995) shall not be used, and mean values of material strengths shall be used in lieu of lower bound estimates.

New walls subjected to lateral forces applied parallel with their plane shall be considered separately from walls subjected to forces applied normal to their plane, as described in Sections 7.4.2 through 7.4.5.

7.4.1.3 Enhanced Masonry Walls

Enhanced masonry walls shall include existing walls that are rehabilitated with the methods given in this section. Unless stated otherwise, methods are applicable to both unreinforced and reinforced walls, and are intended to improve performance of masonry walls subjected to both in-plane and out-of-plane lateral forces.

Enhanced walls subjected to lateral forces applied parallel with their plane shall be considered separately from walls subjected to forces applied normal to their plane, as described in Sections 7.4.2 through 7.4.5.

A. Infilled Openings

An infilled opening shall be considered to act compositely with the surrounding masonry if the following provisions are met.

1. The sum of the lengths of all openings in the direction of in-plane shear force in a single continuous wall is less than 40% of the overall length of the wall.

2. New and old masonry units shall be interlaced at the boundary of the infilled opening with full toothing, or adequate anchorage shall be provided to give an equivalent shear strength at the interface of new and old units.

Stiffness assumptions, strength criteria, and acceptable deformations for masonry walls with infilled openings shall be the same as given for nonrehabilitated solid masonry walls, provided that differences in elastic moduli and strengths for the new and old masonries are considered for the composite section.

B. Enlarged Openings

Openings in a masonry shear wall may be enlarged by removing portions of masonry above or below windows or doors. This is done to increase the height-to-length aspect ratio of piers so that the limit state may be altered from shear to flexure. This method is only applicable to URM walls.

Stiffness assumptions, strength criteria, and acceptable deformations for URM walls with enlarged openings shall be the same as given for existing perforated masonry walls, provided that the cutting operation does not cause any distress.

C. Shotcrete

An existing masonry wall with an application of shotcrete shall be considered to behave as a composite section, as long as adequate anchorage is provided at the shotcrete-masonry interface for shear transfer. Stresses in the masonry and shotcrete shall be determined considering the difference in elastic moduli for each material. Alternatively, the masonry may be neglected if the new shotcrete layer is designed to resist all of the force, and minor cracking of the masonry is acceptable.

Stiffness assumptions, strength criteria, and acceptable deformations for masonry components with shotcrete shall be the same as for new reinforced concrete components, with due consideration to possible variations in boundary conditions.

D. Coatings for URM Walls

A coated masonry wall shall be considered to behave as a composite section, as long as adequate anchorage is provided at the interface between the coating and the masonry wall. Stresses in the masonry and coating shall be determined considering the difference in elastic moduli for each material. If stresses exceed expected strengths of the coating material, then the coating shall be considered ineffective.

Stiffness assumptions, strength criteria, and acceptable deformations for coated masonry walls shall be the same as for existing URM walls.

E. Reinforced Cores for URM Walls

A reinforced-cored masonry wall shall be considered to behave as a reinforced masonry wall, provided that sufficient bonding exists between the new reinforcement and the grout, and between the grout and the cored surface. Vertical reinforcement shall be anchored at the base of the wall to resist its full tensile strength.

Grout in new reinforced cores should consist of cementitious materials whose hardened properties are compatible with those of the surrounding masonry.

Adequate shear strength must exist, or be provided, so that the strength of the new vertical reinforcement can be developed.

Stiffness assumptions, strength criteria, and acceptable deformations for URM walls with reinforced cores shall be the same as for existing reinforced walls.

F. Prestressed Cores for URM Walls

A prestressed-cored masonry wall with unbonded tendons shall be considered to behave as a URM wall with increased vertical compressive stress.

Losses in prestressing force due to creep and shrinkage of the masonry shall be accounted for.

Stiffness assumptions, strength criteria, and acceptable deformations for URM walls with unbonded prestressing tendons shall be the same as for existing unreinforced masonry walls subjected to vertical compressive stress.

G. Grout Injections

Any grout used for filling voids and cracks shall have strength, modulus, and thermal properties compatible with the existing masonry.

Inspection shall be made during the grouting to ensure that voids are completely filled with grout.

Stiffness assumptions, strength criteria, and acceptable deformations for masonry walls with grout injections shall be the same as for existing unreinforced or reinforced walls.

H. Repointing

Bond strength of new mortar shall be equal to or greater than that of the original mortar. Compressive strength of new mortar shall be equal to or less than that of the original mortar.

Stiffness assumptions, strength criteria, and acceptable deformations for repointed masonry walls shall be the same as for existing masonry walls.

I. Braced Masonry Walls

Masonry walls may be braced with external structural elements to reduce span lengths for out-of-plane bending. Adequate strength shall be provided in the bracing element and connections to resist the transfer of forces from the masonry wall to the bracing element. Out-of-plane deflections of braced walls resulting from the transfer of vertical floor or roof loadings shall be considered.

Stiffness assumptions, strength criteria, and acceptable deformations for braced masonry walls shall be the same as for existing masonry walls. Due consideration shall be given to the reduced span of the masonry wall.

J. Stiffening Elements

Masonry walls may be stiffened with external structural members to increase the out-of-plane stiffness and strength. The stiffening member shall be proportioned to resist a tributary portion of lateral load applied normal to the plane of a masonry wall. Adequate connections at the ends of the stiffening element shall be provided to transfer the reaction of force. Flexibility of the stiffening element shall be considered when estimating lateral drift of a masonry wall panel for Performance Levels.

Stiffness assumptions, strength criteria, and acceptable deformations for stiffened masonry walls shall be the same as for existing masonry walls. Due consideration shall be given to the stiffening action that the new element provides.

7.4.2 URM In-Plane Walls and Piers

Information is given in this section for depicting the engineering properties of URM walls subjected to lateral forces applied parallel with their plane. Requirements of this section shall apply to cantilevered shear walls that are fixed against rotation at their base, and piers between window or door openings that are fixed against rotation at their top and base.

Stiffness and strength criteria are presented that are applicable for use with both the Linear Static and Nonlinear Static Procedures prescribed in Chapter 3.

7.4.2.1 Stiffness

The lateral stiffness of masonry wall and pier components shall be determined based on the minimum net sections of mortared and grouted masonry in accordance with the guidelines of this subsection. The lateral stiffness of masonry walls subjected to lateral in-plane forces shall be determined considering both flexural and shear deformations.

The masonry assemblage of units, mortar, and grout shall be considered to be a homogeneous medium for stiffness computations with an expected elastic modulus in compression, E_{me}, as specified in Section 7.3.2.2.

For linear procedures, the stiffness of a URM wall or pier resisting lateral forces parallel with its plane shall be considered to be linear and proportional with the geometrical properties of the uncracked section.

For nonlinear procedures, the in-plane stiffness of URM walls or piers shall be based on the extent of cracking.

Story shears in perforated shear walls shall be distributed to piers in proportion to the relative lateral uncracked stiffness of each pier.

Stiffnesses for existing, enhanced, and new walls shall be determined using the same principles of mechanics.

7.4.2.2 Strength Acceptance Criteria

Unreinforced masonry walls and piers shall be considered as deformation-controlled components if their expected lateral strength limited by bed-joint sliding shear stress or rocking (the lesser of values given by Equations 7-3 and 7-4) is less than the lower bound lateral strength limited by diagonal tension or toe compressive stress (the lesser of values given by Equations 7-5 or 7-6). Otherwise, these components shall be considered as force-controlled components.

A. Expected Lateral Strength of Walls and Piers

Expected lateral strength of existing URM walls or pier components shall be based on expected bed-joint sliding shear strength, or expected rocking strength, in accordance with Equations 7-3 and 7-4, respectively. The strength of such URM walls or piers shall be the lesser of:

$$Q_{CE} = V_{bjs} = v_{me}A_n \tag{7-3}$$

$$Q_{CE} = V_r = 0.9\alpha P_{CE}\left(\frac{L}{h_{eff}}\right) \tag{7-4}$$

where

A_n = Area of net mortared/grouted section, in.^{2}

h_{eff} = Height to resultant of lateral force

L = Length of wall or pier

P_{CE} = Expected vertical axial compressive force per load combinations in Equations 3-2 and 3-3

v_{me} = Expected bed-joint sliding shear strength per Section 7.3.2.4, psi

V_{bjs} = Lateral strength of wall or pier based on bed-joint shear strength, pounds

V_r = Lateral rocking strength of wall or pier component, pounds

α = Factor equal to 0.5 for fixed-free cantilever wall, or equal to 1.0 for fixed-fixed pier

Expected lateral strength of newly constructed wall or pier components shall be based on the *NEHRP Recommended Provisions* (BSSC, 1995), with the exception that capacity reduction factors shall be taken as equal to 1.0.

B. Lower Bound Lateral Strength of Walls and Piers

Lower bound lateral strength of existing URM walls or pier components shall be limited by diagonal tension stress or toe compressive stress, in accordance with Equations 7-5 and 7-6, respectively. The lateral strength of URM walls or piers shall be the lesser of Q_{CL} values given by these two equations.

If L/h_{eff} is larger than 0.67 and less than 1.00, then:

$$Q_{CL} = V_{dt} = f'_{dt} A_n \left(\frac{L}{h_{eff}}\right) \sqrt{1 + \frac{f_a}{f'_{dt}}} \qquad (7\text{-}5)$$

$$Q_{CL} = V_{tc} = \alpha P_{CL} \left(\frac{L}{h_{eff}}\right) \left(1 - \frac{f_a}{0.7 f'_m}\right) \qquad (7\text{-}6)$$

where A_n, h_{eff}, L, and α are the same as given for Equations 7-3 and 7-4, and:

f_a = Upper bound of vertical axial compressive stress from Equation 3-2, psi

f'_{dt} = Lower bound of masonry diagonal tension strength, psi

f'_m = Lower bound of masonry compressive strength, psi

P_{CL} = Lower bound of vertical compressive force from load combination of Equation 3-3, pounds

V_{dt} = Lateral strength limited by diagonal tension stress, pounds

V_{tc} = Lateral strength limited by toe compressive stress, pounds

For determination of V_{dt}, the bed-joint shear strength, v_{me}, may be substituted for the diagonal tension strength, f'_{dt}, in Equation 7-5.

The lower bound masonry compressive strength, f'_m, shall be taken as the expected strength, f_{me}, determined per Section 7.3.2.1, divided by 1.6.

If the Linear Static Procedures of Section 3.3 are used, lateral forces from gravity and seismic effects shall be less than the lower bound lateral strength, Q_{CL}, as required by Equation 3-19.

C. Lower Bound Vertical Compressive Strength of Walls and Piers

Lower bound vertical compressive strength of existing URM walls or pier components shall be limited by masonry compressive stress per Equation 7-7.

$$Q_{CL} = P_c = 0.80(0.85 f'_m A_n) \qquad (7\text{-}7)$$

where f'_m is equal to the expected strength, f_{me}, determined per Section 7.3.2.1, divided by 1.6.

If the Linear Static Procedures of Section 3.3 are used, vertical forces from gravity and seismic effects shall be less than the lower bound lateral strength, Q_{CL}, as stated in Equation 3-19.

7.4.2.3 Deformation Acceptance Criteria

A. Linear Procedures

Table 7-1 Linear Static Procedure—m Factors for URM In-Plane Walls and Piers

Limiting Behavioral Mode	*m* Factors				
	Primary			Secondary	
	IO	LS	CP	LS	CP
Bed-Joint Sliding	1	3	4	6	8
Rocking	$(1.5h_{eff}/L)>1$	$(3h_{eff}/L)>1.5$	$(4h_{eff}/L)>2$	$(6h_{eff}/L)>3$	$(8h_{eff}/L)>4$

Note: Interpolation is permitted between table values.

If the linear procedures of Section 3.3 are used, the product of expected strength, Q_{CE}, of those components classified as deformation-controlled, multiplied by *m* factors given in Table 7-1 for particular Performance Levels and κ factors given in Section 2.7.2, shall exceed the sum of unreduced seismic forces, Q_E, and gravity forces, Q_G, per Equation 3-18. The limiting behavior mode in Table 7-1 shall be identified from the lower of the two expected strengths as determined from Equations 7-3 and 7-4.

For determination of *m* factors from Table 7-1, the vertical compressive stress, f_{ae}, shall be based on an expected value of gravity compressive force given by the load combinations given in Equations 3-2 and 3-3.

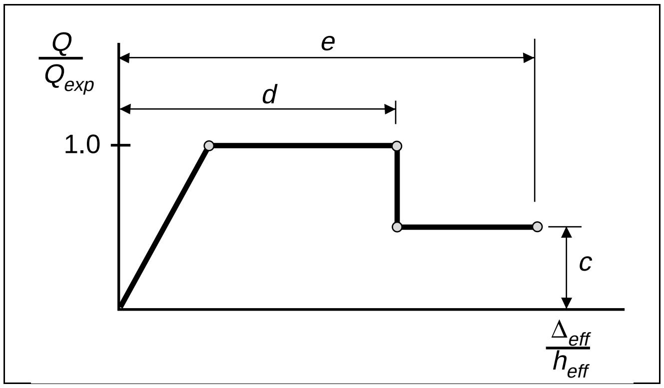

Figure 7-1 Idealized Force-Deflection Relation for Walls, Pier Components, and Infill Panels

B. Nonlinear Procedures

If the Nonlinear Static Procedure given in Section 3.3.3 is used, deformation-controlled wall and pier components shall be assumed to deflect to nonlinear lateral drifts as given in Table 7-2. Variables *d* and *e*, representing nonlinear deformation capacities for primary and secondary components, are expressed in terms of story drift ratio percentages, as defined in Figure 7-1.The limiting behavior mode in Table 7-2 shall be identified from the lower of the two expected strengths as determined from Equations 7-3 and 7-4.

For components of primary lateral-force-resisting elements, collapse shall be considered at lateral drift percentages exceeding values of *d* in the table, and the Life Safety Performance Level shall be considered at approximately 75% of the *d* value. For components of secondary elements, collapse shall be considered at lateral drift percentages exceeding the values of *e* in the table, and the Life Safety Performance Level shall be considered at approximately 75% of the *e* value in the table. Drift percentages based on these criteria are given in Table 7-2.

If the Nonlinear Dynamic Procedure given in Section 3.3.4 is used, nonlinear force-deflection relations for wall and pier components shall be established based on the information given in Table 7-2, or on a more comprehensive evaluation of the hysteretic characteristics of those components.

7.4.3 URM Out-of-Plane Walls

As required by Section 2.11.7, URM walls shall be considered to resist out-of-plane excitation as isolated

Table 7-2 Nonlinear Static Procedure—Simplified Force-Deflection Relations for URM In-Plane Walls and Piers

				Acceptance Criteria				
Limiting Behavioral Mode				**Primary**			**Secondary**	
	c **%**	***d*** **%**	***e*** **%**	**IO %**	**LS %**	**CP %**	**LS %**	**CP %**
Bed-Joint Sliding	0.6	0.4	0.8	0.1	0.3	0.4	0.6	0.8
Rocking	0.6	$0.4h_{eff}/L$	$0.8h_{eff}/L$	0.1	$0.3h_{eff}/L$	$0.4h_{eff}/L$	$0.6h_{eff}/L$	$0.8h_{eff}/L$

Note: Interpolation is permitted between table values.

components spanning between floor levels, and/or spanning horizontally between columns or pilasters. Out-of-plane walls shall not be analyzed with the Linear or Nonlinear Static Procedures prescribed in Chapter 3.

7.4.3.1 Stiffness

The stiffness of out-of-plane walls shall be neglected with analytical models of the global structural system if in-plane walls or infill panels exist, or are placed, in the orthogonal direction.

7.4.3.2 Strength Acceptance Criteria

The Immediate Occupancy Performance Level shall be limited by flexural cracking of out-of-plane walls. Unless arching action is considered, flexural cracking shall be limited by the expected tensile stress values given in Section 7.3.2.3 for existing walls and in BSSC (1995) for new construction.

Arching action shall be considered if, and only if, surrounding floor, roof, column, or pilaster elements have sufficient stiffness and strength to resist thrusts from arching of a wall panel, and a condition assessment has been done to ensure that there are no gaps between a wall panel and the adjacent structure.

Due consideration shall be given to the condition of the collar joint when estimating the effective thickness of a wall.

7.4.3.3 Deformation Acceptance Criteria

The Life Safety and Collapse Prevention Levels permit flexural cracking in URM walls subjected to out-of-plane loading, provided that cracked wall segments will remain stable during dynamic excitation. Stability shall be checked using analytical time-step integration models with realistic depiction of acceleration time histories at the top and base of a wall panel. Walls spanning vertically, with a height-to-thickness (h/t) ratio less than that given in Table 7-3, need not be checked for dynamic stability.

Table 7-3 Permissible h/t Ratios for URM Out-of-Plane Walls

Wall Types	$S_{X1} \leq 0.24g$	$0.24g < S_{X1} \leq 0.37g$	$0.37g < S_{X1} \leq 0.5g$
Walls of one-story buildings	20	16	13
First-story wall of multistory building	20	18	15
Walls in top story of multistory building	14	14	9
All other walls	20	16	13

7.4.4 Reinforced Masonry In-Plane Walls and Piers

7.4.4.1 Stiffness

The stiffness of a reinforced in-plane wall or pier component shall be based on:

- The uncracked section, when an analysis is done to show that the component will not crack when subjected to expected levels of axial and lateral force
- The cracked section, when an analysis is done to show that the component will crack when subjected to expected levels of axial and lateral force

Stiffnesses for existing and new walls shall be assumed to be the same.

7.4.4.2 Strength Acceptance Criteria for Reinforced Masonry (RM)

Strength of RM wall or pier components in flexure, shear, and axial compression shall be determined per the requirements of this section. The assumptions, procedures, and requirements of this section shall apply to both existing and newly constructed RM wall or pier components.

Reinforced masonry walls and piers shall be considered as deformation-controlled components if their expected lateral strength for flexure per Section 7.4.4.2A is less than the lower bound lateral strength limited by shear per Section 7.4.4.2B. Vertical compressive behavior of reinforced masonry wall or pier components shall be assumed to be a force-controlled action. Methods for determining lower bound axial compressive strength are given in Section 7.4.4.2D.

A. Expected Flexural Strength of Walls and Piers

Expected flexural strength of an RM wall or pier shall be determined on the basis of the following assumptions.

- Stress in reinforcement below the expected yield strength, f_{ye}, shall be taken as the modulus of elasticity, E_{se}, times the steel strain. For reinforcement strains larger than those corresponding to the expected yield strength, the stress in the reinforcement shall be considered independent of strain and equal to the expected yield strength, f_{ye}.

- Tensile strength of masonry shall be neglected in calculating the flexural strength of a reinforced masonry cross section.

- Flexural compression stress in masonry shall be assumed to be distributed across an equivalent rectangular stress block. Masonry stress of 0.85 times the expected compressive strength, f_{me}, shall be distributed uniformly over an equivalent compression zone bounded by edges of the cross section and with a depth equal to 85% of the depth from the neutral axis to the fiber of maximum compressive strain.

- Strains in the reinforcement and masonry shall be considered linear across the wall or pier cross section. For purposes of determining forces in reinforcing bars distributed across the section, the maximum compressive strain in the masonry shall be assumed to be equal to 0.003.

B. Lower-Bound Shear Strength of Walls and Piers

Lower-bound shear strength of RM wall or pier components, V_{CL}, shall be determined using Equation 7-8.

$$Q_{CL} = V_{CL} = V_{mL} + V_{sL} \tag{7-8}$$

where:

V_{mL} = Lower bound shear strength provided by masonry, lb
V_{sL} = Lower bound shear strength provided by reinforcement, lb

The lower bound shear strength of an RM wall or pier shall not exceed shear forces given by Equations 7-9 and 7-10.

For M/Vd_v less than 0.25:

$$V_{CL} \le 6\sqrt{f'_m}A_n \tag{7-9}$$

For M/Vd_v greater than or equal to 1.00:

$$V_{CL} \le 4\sqrt{f'_m}A_n \tag{7-10}$$

where:

A_n = Area of net mortared/grouted section, in.^{2}
f'_m = Compressive strength of masonry, psi
M = Moment on the masonry section, in.-lb
V = Shear on the masonry section, lb
d_v = Wall length in direction of shear force, in.

Lower-bound shear strength, V_{mL}, resisted by the masonry shall be determined using Equation 7-11.

$$V_{mL} = \left[4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right] A_n \sqrt{f_m} + 0.25 P_{CL} \quad (7\text{-}11)$$

where M/Vd_v need not be taken greater than 1.0, and P_{CL} is the lower-bound vertical compressive force in pounds based on the load combinations given in Equations 3-2 and 3-3.

Lower-bound shear strength, V_{sL}, resisted by the reinforcement shall be determined using Equation 7-12.

$$V_{sL} = 0.5\left(\frac{A_v}{s}\right) f_y d_v \quad (7\text{-}12)$$

where:

A_v = Area of shear reinforcement, in.^{2}

s = Spacing of shear reinforcement, in.

f_y = Lower-bound yield strength of shear reinforcement, psi

C. Strength Considerations for Flanged Walls

Wall intersections shall be considered effective in transferring shear when either condition (1) or (2), and condition (3), as noted below, are met:

1. The face shells of hollow masonry units are removed and the intersection is fully grouted.
2. Solid units are laid in running bond, and 50% of the masonry units at the intersection are interlocked.
3. Reinforcement from one intersecting wall continues past the intersection a distance not less than 40 bar diameters or 24 inches.

The width of flange considered effective in compression on each side of the web shall be taken as equal to six times the thickness of the web, or shall be equal to the actual flange on either side of the web wall, whichever is less.

The width of flange considered effective in tension on each side of the web shall be taken as equal to 3/4 of the wall height, or shall be equal to the actual flange on either side of the web wall, whichever is less.

D. Lower Bound Vertical Compressive Strength of Walls and Piers

Lower bound vertical compressive strength of existing RM walls or pier components shall be determined using Equation 7-13.

$$Q_{CL} = P_c = 0.8[0.85 f'_m (A_n - A_s) + A_s f_y] \quad (7\text{-}13)$$

where:

f'_m = Lower bound masonry compressive strength equal to expected strength, f_{me}, determined per Section 7.3.2.1, divided by 1.6

f_y = Lower bound reinforcement yield strength per Section 7.3.2.6

If the Linear Static Procedures of Section 3.3.1 are used, vertical forces from gravity and seismic effects shall be less than the lower bound lateral strength, Q_{CL}, as required by Equation 3-19.

7.4.4.3 Deformation Acceptance Criteria

A. Linear Procedures

If the linear procedures of Section 3.3 are used, the product of expected strength, Q_{CE}, of those components classified as deformation-controlled, multiplied by m factors given in Table 7-4 for particular Performance Levels and κ factors given in Section 2.7.2, shall exceed the sum, Q_{UD},of unreduced seismic forces, Q_E, and gravity forces, Q_G, as in Equations 3-14 and 3-18.

For determination of m factors from Table 7-4, the ratio of vertical compressive stress to expected compressive strength, f_{ae}/f_{me}, shall be based on an expected value of gravity compressive force per the load combinations given in Equations 3-2 and 3-3.

B. Nonlinear Procedures

If the Nonlinear Static Procedure given in Section 3.3.3 is used, deformation-controlled wall and pier components shall be assumed to deflect to nonlinear lateral drifts as given in Table 7-5. Variables d and e, representing nonlinear deformation capacities for primary and secondary components, are expressed in terms of story drift ratio percentages as defined in Figure 7-1.

For determination of the c, d, and e values and the acceptable drift levels using Table 7-5, the vertical compressive stress, f_{ae}, shall be based on an expected value of gravity compressive force per the load combinations given in Equations 3-2 and 3-3.

For components of primary lateral-force-resisting elements, collapse shall be considered at lateral drift percentages exceeding values of *d* in Table 7-5, and the Life Safety Performance Level shall be considered at approximately 75% of the *d* value. For components of secondary elements, collapse shall be considered at lateral drift percentages exceeding the values of *e* in the table, and the Life Safety Performance Level shall be considered at approximately 75% of the *e* value in the table. Story drift ratio percentages based on these criteria are given in Table 7-5.

If the Nonlinear Dynamic Procedure given in Section 3.3.4 is used, nonlinear force-deflection relations for wall and pier components shall be established based on the information given in Table 7-5, or on a more comprehensive evaluation of the hysteretic characteristics of those components.

Acceptable deformations for existing and new walls shall be assumed to be the same.

7.4.5 RM Out-of-Plane Walls

As required by Section 2.11.7, RM walls shall be considered to resist out-of-plane excitation as isolated components spanning between floor levels, and/or spanning horizontally between columns or pilasters. Out-of-plane walls shall not be analyzed with the Linear or Nonlinear Static Procedures prescribed in Chapter 3, but shall resist lateral inertial forces as given in Section 2.11.7, or respond to earthquake motions as determined with the Nonlinear Dynamic Procedure and satisfy the deflection criteria given in Section 7.4.5.3.

7.4.5.1 Stiffness

Out-of-plane RM walls shall be considered as local elements spanning between individual story levels.

The stiffness of out-of-plane walls shall be neglected with analytical models of the global structural system if in-plane walls exist or are placed in the orthogonal direction.

Uncracked sections based on the net mortared/grouted area shall be considered for determination of geometrical properties, provided that net flexural tensile stress does not exceed expected tensile strength, f_{te}, per Section 7.3.2.3. Stiffness shall be based on a cracked section for a wall whose net flexural tensile stress exceeds expected tensile strength.

Stiffnesses for existing and new reinforced out-of-plane walls shall be assumed to be the same.

7.4.5.2 Strength Acceptance Criteria

Out-of-plane RM walls shall be sufficiently strong in flexure to resist the transverse loadings prescribed in Section 2.11.7 for all Performance Levels. Expected flexural strength shall be based on the assumptions given in Section 7.4.4.2A. For walls with an h/t ratio exceeding 20, the effects of deflections on moments shall be considered.

Strength of new walls and existing walls shall be assumed to be the same.

7.4.5.3 Deformation Acceptance Criteria

If the Nonlinear Dynamic Procedure is used, the following performance criteria shall be based on the maximum deflection normal to the plane of a transverse wall.

- The Immediate Occupancy Performance Level shall be met when significant visual cracking of an RM wall occurs. This limit state shall be assumed to occur at a lateral story drift ratio of approximately 2%.
- The Life Safety Performance Level shall be met when masonry units are dislodged and fall out of the wall. This limit state shall be assumed to occur at a lateral drift of a story panel equal to approximately 3%.
- The Collapse Prevention Performance Level shall be met when the post-earthquake damage state is on the verge of collapse. This limit state shall be assumed to occur at a lateral story drift ratio of approximately 5%.

Acceptable deformations for existing and new walls shall be assumed to be the same.

7.5 Engineering Properties of Masonry Infills

This section provides basic engineering information for assessing attributes of masonry infill panels, including

Table 7-4 Linear Static Procedure—m Factors for Reinforced Masonry In-Plane Walls

f_{ae}/f_{me}	L/h_{eff}	$\rho_g f_{ye}/f_{me}$	*m* Factors				
			Primary			Secondary	
			IO	LS	CP	LS	CP
0.00	0.5	0.01	4.0	7.0	8.0	8.0	10.0
		0.05	2.5	5.0	6.5	8.0	10.0
		0.20	1.5	2.0	2.5	4.0	5.0
	1.0	0.01	4.0	7.0	8.0	8.0	10.0
		0.05	3.5	6.5	7.5	8.0	10.0
		0.20	1.5	3.0	4.0	6.0	8.0
	2.0	0.01	4.0	7.0	8.0	8.0	10.0
		0.05	3.5	6.5	7.5	8.0	10.0
		0.20	2.0	3.5	4.5	7.0	9.0
0.038	0.5	0.01	3.0	6.0	7.5	8.0	10.0
		0.05	2.0	3.5	4.5	7.0	9.0
		0.20	1.5	2.0	2.5	4.0	5.0
	1.0	0.01	4.0	7.0	8.0	8.0	10.0
		0.05	2.5	5.0	6.5	8.0	10.0
		0.20	1.5	2.5	3.5	5.0	7.0
	2.0	0.01	4.0	7.0	8.0	8.0	10.0
		0.05	3.5	6.5	7.5	8.0	10.0
		0.20	1.5	3.0	4.0	6.0	8.0
0.075	0.5	0.01	2.0	3.5	4.5	7.0	9.0
		0.05	1.5	3.0	4.0	6.0	8.0
		0.20	1.0	2.0	2.5	4.0	5.0
	1.0	0.01	2.5	5.0	6.5	8.0	10.0
		0.05	2.0	3.5	4.5	7.0	9.0
		0.20	1.5	2.5	3.5	5.0	7.0
	2.0	0.01	4.0	7.0	8.0	8.0	10.0
		0.05	2.5	5.0	6.5	8.0	10.0
		0.20	1.5	3.0	4.0	4.0	8.0

Note: Interpolation is permitted between table values.

stiffness assumptions, and the strength acceptance and deformation acceptance criteria for the Immediate Occupancy, Life Safety, and Collapse Prevention Performance Levels. Engineering properties given for masonry infills shall be used with the analytical methods prescribed in Chapter 3, unless noted otherwise.

Masonry infill panels shall be considered as primary elements of a lateral-force-resisting system. If the surrounding frame can remain stable following the loss of an infill panel, infill panels shall not be subject to limits set by the Collapse Prevention Performance Level.

Table 7-5 Nonlinear Static Procedure—Simplified Force-Deflection Relations for Reinforced Masonry Shear Walls

						Acceptance Criteria				
						Primary			Secondary	
f_{ae}/f_{me}	L/h_{eff}	$\rho_g f_{ye}/f_{me}$	c	d %	e %	IO %	LS %	CP %	LS %	CP %
0.00	0.5	0.01	0.5	2.6	5.3	1.0	2.0	2.6	3.9	5.3
		0.05	0.6	1.1	2.2	0.4	0.8	1.1	1.6	2.2
		0.20	0.7	0.5	1.0	0.2	0.4	0.5	0.7	1.0
	1.0	0.01	0.5	2.1	4.1	0.8	1.6	2.1	3.1	4.1
		0.05	0.6	0.8	1.6	0.3	0.6	0.8	1.2	1.6
		0.20	0.7	0.3	0.6	0.1	0.2	0.3	0.5	0.6
	2.0	0.01	0.5	1.6	3.3	0.6	1.2	1.6	2.5	3.3
		0.05	0.6	0.6	1.3	0.2	0.5	0.6	0.9	1.3
		0.20	0.7	0.2	0.4	0.1	0.2	0.2	0.3	0.4
0.038	0.5	0.01	0.4	1.0	2.0	0.4	0.8	1.0	1.5	2.0
		0.05	0.5	0.7	1.4	0.3	0.5	0.7	1.0	1.4
		0.20	0.6	0.4	0.9	0.2	0.3	0.4	0.7	0.9
	1.0	0.01	0.4	0.8	1.5	0.3	0.6	0.8	1.1	1.5
		0.05	0.5	0.5	1.0	0.2	0.4	0.5	0.7	1.0
		0.20	0.6	0.3	0.6	0.1	0.2	0.3	0.4	0.6
	2.0	0.01	0.4	0.6	1.2	0.2	0.4	0.6	0.9	1.2
		0.05	0.5	0.4	0.7	0.1	0.3	0.4	0.5	0.7
		0.20	0.6	0.2	0.4	0.1	0.1	0.2	0.3	0.4
0.075	0.5	0.01	0.3	0.6	1.2	0.2	0.5	0.6	0.9	1.2
		0.05	0.4	0.5	1.0	0.2	0.4	0.5	0.8	1.0
		0.20	0.5	0.4	0.8	0.1	0.3	0.4	0.6	0.8
	1.0	0.01	0.3	0.4	0.9	0.2	0.3	0.4	0.7	0.9
		0.05	0.4	0.4	0.7	0.1	0.3	0.4	0.5	0.7
		0.20	0.5	0.2	0.5	0.1	0.2	0.2	0.4	0.5
	2.0	0.01	0.3	0.3	0.7	0.1	0.2	0.3	0.5	0.7
		0.05	0.4	0.3	0.5	0.1	0.2	0.3	0.4	0.5
		0.20	0.5	0.2	0.3	0.1	0.1	0.2	0.2	0.3

Note: Interpolation is permitted between table values.

7.5.1 Types of Masonry Infills

Procedures set forth in this section are applicable to existing panels, panels enhanced for seismic rehabilitation, and new panels added to an existing frame. Infills shall include panels built partially or fully within the plane of steel or concrete frames, and bounded by beams and columns around their perimeters.

Infill panel types considered in these *Guidelines* include unreinforced clay-unit masonry, concrete masonry, and hollow-clay tile masonry. Infills made of stone or glass block are not addressed.

Infill panels that are considered to be isolated from the surrounding frame must have sufficient gaps at top and sides to accommodate maximum lateral frame deflections. Isolated panels shall be restrained in the transverse direction to insure stability under normal forces. Panels that are in tight contact with the frame elements on all four sides are termed shear infill panels. For panels to be considered under this designation, any gaps between an infill and a surrounding frame shall be filled to provide tight contact.

Frame members and connections surrounding infill panels shall be evaluated for frame-infill interaction effects. These effects shall include forces transferred from an infill panel to beams, columns, and connections, and bracing of frame members across a partial length.

7.5.1.1 Existing Masonry Infills

Existing masonry infills considered in this section shall include all structural infills of a building system that are in place prior to seismic rehabilitation.

Infill types included in this section consist of unreinforced and ungrouted panels, and composite or noncomposite panels. Existing infill panels subjected to lateral forces applied parallel with their plane shall be considered separately from infills subjected to forces applied normal to their plane, as described in Sections 7.5.2 and 7.5.3.

Material properties for existing infills shall be established per Section 7.3.2. Prior to rehabilitation, masonry infills shall be assessed for condition per procedures set forth in Sections 7.3.3.1, 7.3.3.2, or 7.3.3.3. Existing masonry infills shall be assumed to behave the same as new masonry infills, provided that a condition assessment demonstrates equivalent quality of construction.

7.5.1.2 New Masonry Infills

New masonry infills shall include all new panels added to an existing lateral-force-resisting system for structural rehabilitation. Infill types shall include unreinforced, ungrouted, reinforced, grouted and partially grouted, and composite or noncomposite.

When analyzing a system of new infills, expected values of strength and stiffness shall be used. No capacity reduction factors shall be used, and expected values of material strengths shall be used in lieu of lower bound estimates.

7.5.1.3 Enhanced Masonry Infills

Enhanced masonry infill panels shall include existing infills that are rehabilitated with the methods given in this section. Unless stated otherwise, methods are applicable to unreinforced infills, and are intended to improve performance of masonry infills subjected to both in-plane and out-of-plane lateral forces.

Masonry infills that are enhanced in accordance with the minimum standards of this section shall be considered using the same Analysis Procedures and performance criteria as for new infills.

Guidelines from the following sections, pertaining to enhancement methods for unreinforced masonry walls, shall also apply to unreinforced masonry infill panels: (1) “Infilled Openings,” Section 7.4.1.3A; (2) “Shotcrete,” Section 7.4.1.3C; (3) “Coatings for URM Walls,” Section 7.4.1.3D; (4) “Grout Injections,” Section 7.4.1.3G; (5) “Repointing,” Section 7.4.1.3H; and (6) “Stiffening Elements,” Section 7.4.1.3J. In addition, the following two enhancement methods shall also apply to masonry infill panels.

A. Boundary Restraints for Infill Panels

Infill panels not in tight contact with perimeter frame members shall be restrained for out-of-plane forces. This may be accomplished by installing steel angles or plates on each side of the infills, and welding or bolting the angles or plates to the perimeter frame members.

B. Joints Around Infill Panels

Gaps between an infill panel and the surrounding frame shall be filled if integral infill-frame action is assumed for in-plane response.

7.5.2 In-Plane Masonry Infills

7.5.2.1 Stiffness

The elastic in-plane stiffness of a solid unreinforced masonry infill panel prior to cracking shall be represented with an equivalent diagonal compression strut of width, a, given by Equation 7-14. The equivalent strut shall have the same thickness and modulus of elasticity as the infill panel it represents.

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf} \qquad (7\text{-}14)$$

where

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4 E_{fe} I_{col} h_{inf}}\right]^{\frac{1}{4}}$$

and

- h_{col} = Column height between centerlines of beams, in.
- h_{inf} = Height of infill panel, in.
- E_{fe} = Expected modulus of elasticity of frame material, psi
- E_{me} = Expected modulus of elasticity of infill material, psi
- I_{col} = Moment of inertia of column, in.^{4}
- L_{inf} = Length of infill panel, in.
- r_{inf} = Diagonal length of infill panel, in.
- t_{inf} = Thickness of infill panel and equivalent strut, in.
- θ = Angle whose tangent is the infill height-to-length aspect ratio, radians
- λ_1 = Coefficient used to determine equivalent width of infill strut

For noncomposite infill panels, only the wythes in full contact with the frame elements shall be considered when computing in-plane stiffness, unless positive anchorage capable of transmitting in-plane forces from frame members to all masonry wythes is provided on all sides of the walls.

Stiffness of cracked unreinforced masonry infill panels shall be represented with equivalent struts, provided that the strut properties are determined from detailed analyses that consider the nonlinear behavior of the infilled frame system after the masonry is cracked.

The equivalent compression strut analogy shall be used to represent the elastic stiffness of a perforated unreinforced masonry infill panel, provided that the equivalent strut properties are determined from appropriate stress analyses of infill walls with representative opening patterns.

Stiffnesses for existing and new infills shall be assumed to be the same.

7.5.2.2 Strength Acceptance Criteria

A. Infill Shear Strength

The transfer of story shear across a masonry infill panel confined within a concrete or steel frame shall be considered as a deformation-controlled action. Expected in-plane panel shear strength shall be determined per the requirements of this section.

Expected infill shear strength, V_{ine}, shall be calculated as the product of the net mortared and grouted area of the infill panel, A_{ni}, times the expected shear strength of the masonry, f_{vie}, in accordance with Equation 7-15.

$$Q_{CE} = V_{ine} = A_{ni} f_{vie} \qquad (7\text{-}15)$$

where:

- A_{ni} = Area of net mortared/grouted section across infill panel, in.^{2}
- f_{vie} = Expected shear strength of masonry infill, psi

Expected shear strength of existing infills, f_{vie}, shall be taken to not exceed the expected masonry bed-joint shear strength, v_{me}, as determined per Section 7.3.2.4.

Shear strength of newly constructed infill panels, f_{vie}, shall not exceed values given in Section 8.7.4 of BSSC (1995) for zero vertical compressive stress.

For noncomposite infill panels, only the wythes in full contact with the frame elements shall be considered when computing in-plane strength, unless positive anchorage capable of transmitting in-plane forces from frame members to all masonry wythes is provided on all sides of the walls.

B. Required Strength of Column Members Adjacent to Infill Panels

Unless a more rigorous analysis is done, the expected flexural and shear strengths of column members adjacent to an infill panel shall exceed forces resulting from one of the following conditions:

1. The application of the horizontal component of the expected infill strut force applied at a distance, l_{ceff}, from the top or bottom of the infill panel equal to:

$$l_{ceff} = \frac{a}{\cos\theta_c} \tag{7-16}$$

where:

$$\tan\theta_c = \frac{h_{inf} - \frac{a}{\cos\theta_c}}{L_{inf}} \tag{7-17}$$

2. The shear force resulting from development of expected column flexural strengths at the top and bottom of a column with a reduced height equal to l_{ceff}

The reduced column length, l_{ceff}, in Equation 7-16 shall be equal to the clear height of opening for a captive column braced laterally with a partial height infill.

The requirements of this section shall be waived if the expected masonry shear strength, v_{me}, as measured per test procedures of Section 7.3.2.4, is less than 50 psi.

C. Required Strength of Beam Members Adjacent to Infill Panels

The expected flexural and shear strengths of beam members adjacent to an infill panel shall exceed forces resulting from one of the following conditions:

1. The application of the vertical component of the expected infill strut force applied at a distance, l_{beff}, from the top or bottom of the infill panel equal to:

$$l_{ceff} = \frac{a}{\sin\theta_b} \tag{7-18}$$

where:

$$\tan\theta_b = \frac{h_{inf}}{L_{inf} - \frac{a}{\sin\theta_b}} \tag{7-19}$$

2. The shear force resulting from development of expected beam flexural strengths at the ends of a beam member with a reduced length equal to l_{beff}

The requirements of this section shall be waived if the expected masonry shear strength, v_{me}, as measured per test procedures of Section 7.3.2.4, is less than 50 psi.

7.5.2.3 Deformation Acceptance Criteria

A. Linear Procedures

If the linear procedures of Section 3.3.1 are used, the product of expected infill strength, V_{ine}, multiplied by m factors given in Table 7-6 for particular Performance Levels and κ factors given in Section 2.7.2, shall exceed the sum of unreduced seismic forces, Q_E, and gravity forces, Q_G, per Equation 3-18. For the case of an infill panel, Q_E shall be the horizontal component of the unreduced axial force in the equivalent strut member.

For determination of m factors per Table 7-6, the ratio of frame to infill strengths shall be determined considering the expected lateral strength of each component. If the expected frame strength is less than 0.3 times the expected infill strength, the confining effects of the frame shall be neglected and the masonry component shall be evaluated as an individual wall component per Sections 7.4.2 or 7.4.4.

B. Nonlinear Procedures

If the Nonlinear Static Procedure given in Section 3.3.3 is used, infill panels shall be assumed to deflect to nonlinear lateral drifts as given in Table 7-7. The variable d, representing nonlinear deformation capacities, is expressed in terms of story drift ratio in percent as defined in Figure 7-1.

For determination of the d values and the acceptable drift levels using Table 7-7, the ratio of frame to infill strengths shall be determined considering the expected lateral strength of each component. If the expected frame strength is less than 0.3 times the expected infill strength, the confining effects of the frame shall be neglected and the masonry component shall be

Table 7-6 Linear Static Procedure—m Factors for Masonry Infill Panels

$\beta = \frac{V_{fre}}{V_{ine}}$	$\frac{L_{inf}}{h_{inf}}$	*m* Factors		
		IO	LS	CP
$0.3 \leq \beta < 0.7$	0.5	1.0	4.0	n.a.
	1.0	1.0	3.5	n.a.
	2.0	1.0	3.0	n.a.
$0.7 \leq \beta < 1.3$	0.5	1.5	6.0	n.a.
	1.0	1.2	5.2	n.a.
	2.0	1.0	4.5	n.a.
$\beta \geq 1.3$	0.5	1.5	8.0	n.a.
	1.0	1.2	7.0	n.a.
	2.0	1.0	6.0	n.a.

Note: Interpolation is permitted between table values.

evaluated as an individual wall component per Sections 7.4.2 or 7.4.4.

The Immediate Occupancy Performance Level shall be met when significant visual cracking of an unreinforced masonry infill occurs. The Life Safety Performance Level shall be met when substantial cracking of the masonry infill occurs, and the potential for the panel, or some portion of it, to drop out of the frame is high. Acceptable story drift ratio percentages corresponding to these general Performance Levels are given in Table 7-7.

If the surrounding frame can remain stable following the loss of an infill panel, infill panels shall not be subject to limits set by the Collapse Prevention Performance Level.

If the Nonlinear Dynamic Procedure given in Section 3.3.4 is used, nonlinear force-deflection relations for infill panels shall be established based on the information given in Table 7-7, or on a more comprehensive evaluation of the hysteretic characteristics of those components.

Acceptable deformations for existing and new infills shall be assumed to be the same.

Table 7-7 Nonlinear Static Procedure—Simplified Force-Deflection Relations for Masonry Infill Panels

$\beta = \frac{V_{fre}}{V_{ine}}$	$\frac{L_{inf}}{h_{inf}}$	c	d %	e %	Acceptance Criteria LS %	Acceptance Criteria CP %
$0.3 \leq \beta < 0.7$	0.5	n.a.	0.5	n.a.	0.4	n.a.
	1.0	n.a.	0.4	n.a.	0.3	n.a.
	2.0	n.a.	0.3	n.a.	0.2	n.a.
$0.7 \leq \beta < 1.3$	0.5	n.a.	1.0	n.a.	0.8	n.a.
	1.0	n.a.	0.8	n.a.	0.6	n.a.
	2.0	n.a.	0.6	n.a.	0.4	n.a.
$\beta \geq 1.3$	0.5	n.a.	1.5	n.a.	1.1	n.a.
	1.0	n.a.	1.2	n.a.	0.9	n.a.
	2.0	n.a.	0.9	n.a.	0.7	n.a.

Note: Interpolation is permitted between table values.

7.5.3 Out-of-Plane Masonry Infills

Unreinforced infill panels with h_{inf}/t_{inf} ratios less than those given in Table 7-8, and meeting the requirements for arching action given in the following section, need not be analyzed for transverse seismic forces.

7.5.3.1 Stiffness

Out-of-plane infill panels shall be considered as local elements spanning vertically between floor levels and/or horizontally across bays of frames.

Table 7-8 ***Maximum h_{inf}/t_{inf} Ratios for which No Out-of-Plane Analysis is Necessary***

	Low Seismic Zone	Moderate Seismic Zone	High Seismic Zone
IO	14	13	8
LS	15	14	9
CP	16	15	10

The stiffness of infill panels subjected to out-of-plane forces shall be neglected with analytical models of the global structural system if in-plane walls or infill panels exist, or are placed, in the orthogonal direction.

Flexural stiffness for uncracked masonry infills subjected to transverse forces shall be based on the minimum net sections of mortared and grouted masonry. Flexural stiffness for unreinforced, cracked infills subjected to transverse forces shall be assumed to be equal to zero unless arching action is considered.

Arching action shall be considered if, and only if, the following conditions exist.

- The panel is in tight contact with the surrounding frame components.
- The product of the elastic modulus, E_{fe}, times the moment of inertia, I_f, of the most flexible frame component exceeds a value of 3.6 x 10^9 lb-in.^{2}.
- The frame components have sufficient strength to resist thrusts from arching of an infill panel.
- The h_{inf}/t_{inf} ratio is less than or equal to 25.

For such cases, mid-height deflection normal to the plane of an infill panel, Δ_{inf}, divided by the infill height, h_{inf}, shall be determined in accordance with Equation 7-20.

$$\frac{\Delta_{inf}}{h_{inf}} = \frac{0.002\left(\frac{h_{inf}}{t_{inf}}\right)}{1 + \sqrt{1 - 0.002\left(\frac{h_{inf}}{t_{inf}}\right)^2}} \tag{7-20}$$

For infill panels not meeting the requirements for arching action, deflections shall be determined with the procedures given in Sections 7.4.3 or 7.4.5.

Stiffnesses for existing and new infills shall be assumed to be the same.

7.5.3.2 Strength Acceptability Criteria

Masonry infill panels shall resist out-of-plane inertial forces as given in Section 2.11.7. Transversely loaded infills shall not be analyzed with the Linear or Nonlinear Static Procedures prescribed in Chapter 3.

The lower bound transverse strength of a URM infill panel shall exceed normal pressures as prescribed in Section 2.11.7. Unless arching action is considered, the lower bound strength of a URM infill panel shall be limited by the lower bound masonry flexural tension strength, f'_t, which may be taken as 0.7 times the expected tensile strength, f_{te}, as determined per Section 7.3.2.3.

Arching action shall be considered only when the requirements stated in the previous section are met. In such case, the lower bound transverse strength of an infill panel in pounds per square foot, q_{in}, shall be determined using Equation 7-21.

$$Q_{CL} = q_{in} = \frac{0.7 f'_m \lambda_2}{\left(\frac{h_{inf}}{t_{inf}}\right)} \times 144 \tag{7-21}$$

where

f'_m = Lower bound of masonry compressive strength equal to $f_{me}/1.6$, psi

λ_2 = Slenderness parameter as defined in Table 7-9

Table 7-9 ***Values of λ_2 for Use in Equation 7-21***

h_{inf}/t_{inf}	5	10	15	25
λ_2	0.129	0.060	0.034	0.013

7.5.3.3 Deformation Acceptance Criteria

The Immediate Occupancy Performance Level shall be met when significant visual cracking of a URM infill occurs. This limit state shall be assumed to occur at an out-of-plane drift of a story panel equal to approximately 2%.

The Life Safety Performance Level shall be met when substantial damage to a URM infill occurs, and the potential for the panel, or some portion of it, to drop out of the frame is high. This limit state shall be assumed to occur at an out-of-plane lateral story drift ratio equal to approximately 3%.

If the surrounding frame can remain stable following the loss of an infill panel, infill panels shall not be subject to limits set by the Collapse Prevention Performance Level.

Acceptable deformations of existing and new walls shall be assumed to be the same.

7.6 Anchorage to Masonry Walls

7.6.1 Types of Anchors

Anchors considered in Section 7.6.2 shall include plate anchors, headed anchor bolts, and bent bar anchor bolts embedded into clay-unit and concrete masonry. Anchors in hollow-unit masonry must be embedded in grout.

Pullout and shear strength of expansion anchors shall be verified by tests.

7.6.2 Analysis of Anchors

Anchors embedded into existing or new masonry walls shall be considered as force-controlled components. Lower bound values for strengths with respect to pullout, shear, and combinations of pullout and shear, shall be based on Section 8.3.12 of BSSC (1995) for embedded anchors.

The minimum effective embedment length for considerations of pullout strength shall be as defined in Section 8.3.12 of BSSC (1995). When the embedment length is less than four bolt diameters or two inches (50.8 mm), the pullout strength shall be taken as zero.

The minimum edge distance for considerations of full shear strength shall be 12 bar diameters. Shear strength of anchors with edge distances equal to or less than one inch (25.4 mm) shall be taken as zero. Linear interpolation of shear strength for edge distances between these two bounds is permitted.

7.7 Masonry Foundation Elements

7.7.1 Types of Masonry Foundations

Masonry foundations are common in older buildings and are still used for some modern construction. Such foundations may include footings and foundation walls constructed of stone, clay brick, or concrete block. Generally, masonry footings are unreinforced; foundation walls may or may not be reinforced.

Spread footings transmit vertical column and wall loads to the soil by direct bearing. Lateral forces are transferred through friction between the soil and the masonry, as well as by passive pressure of the soil acting on the vertical face of the footing.

7.7.2 Analysis of Existing Foundations

A lateral-force analysis of a building system shall include the deformability of the masonry footings, and the flexibility of the soil under them. The strength and stiffness of the soil shall be checked per Section 4.4.

Masonry footings shall be modeled as elastic components with little or no inelastic deformation capacity, unless verification tests are done to prove otherwise. For the Linear Static Procedure, masonry footings shall be considered to be force-controlled components (m equals 1.0).

Masonry retaining walls shall resist active and passive soil pressures per Section 4.5. Stiffness, and strength and acceptability criteria for masonry retaining walls shall be the same as for other masonry walls subjected to transverse loadings, as addressed in Sections 7.4.3 and 7.4.5.

7.7.3 Rehabilitation Measures

In addition to those rehabilitation measures provided for concrete foundation elements in Section 6.13.4, masonry foundation elements may also be rehabilitated with the following options:

- Injection grouting of stone foundations
- Reinforcing of URM foundations

- Prestressing of masonry foundations
- Enlargement of footings by placement of reinforced shotcrete
- Enlargement of footings with additional reinforced concrete sections

Procedures for rehabilitation shall follow provisions for enhancement of masonry walls where applicable per Section 7.4.1.3.

7.8 Definitions

Bearing wall: A wall that supports gravity loads of at least 200 pounds per linear foot from floors and/or roofs.

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

Cavity wall: A masonry wall with an air space between wythes. Wythes are usually joined by wire reinforcement, or steel ties. Also known as a noncomposite wall.

Clay-unit masonry: Masonry constructed with solid, cored, or hollow units made of clay. Hollow clay units may be ungrouted, or grouted.

Clay tile masonry: Masonry constructed with hollow units made of clay tile. Typically, units are laid with cells running horizontally, and are thus ungrouted. In some cases, units are placed with cells running vertically, and may or may not be grouted.

Collar joint: Vertical longitudinal joint between wythes of masonry or between masonry wythe and back-up construction that may be filled with mortar or grout.

Composite masonry wall: Multiwythe masonry wall acting with composite action.

Concrete masonry: Masonry constructed with solid or hollow units made of concrete. Hollow concrete units may be ungrouted, or grouted.

Head joint: Vertical mortar joint placed between masonry units in the same wythe.

Hollow masonry unit: A masonry unit whose net cross-sectional area in every plane parallel to the bearing surface is less than 75% of the gross cross-sectional area in the same plane.

Infill: A panel of masonry placed within a steel or concrete frame. Panels separated from the surrounding frame by a gap are termed “isolated infills.” Panels that are in tight contact with a frame around its full perimeter are termed “shear infills.”

In-plane wall: See shear wall.

Masonry: The assemblage of masonry units, mortar, and possibly grout and/or reinforcement. Types of masonry are classified herein with respect to the type of the masonry units, such as clay-unit masonry, concrete masonry, or hollow-clay tile masonry.

Nonbearing wall: A wall that supports gravity loads less than as defined for a bearing wall.

Noncomposite masonry wall: Multiwythe masonry wall acting without composite action.

Out-of-plane wall: A wall that resists lateral forces applied normal to its plane.

Parapet: Portions of a wall extending above the roof diaphragm. Parapets can be considered as flanges to roof diaphragms if adequate connections exist or are provided.

Partially grouted masonry wall: A masonry wall containing grout in some of the cells.

Perforated wall or infill panel: A wall or panel not meeting the requirements for a solid wall or infill panel.

Pier: A vertical portion of masonry wall between two horizontally adjacent openings. Piers resist axial stresses from gravity forces, and bending moments from combined gravity and lateral forces.

Reinforced masonry (RM) wall: A masonry wall that is reinforced in both the vertical and horizontal directions. The sum of the areas of horizontal and vertical reinforcement must be at least 0.002 times the gross cross-sectional area of the wall, and the minimum area of reinforcement in each direction must be not less than 0.0007 times the gross cross-sectional area of the wall. Reinforced walls are assumed to resist loads through resistance of the masonry in compression and the reinforcing steel in tension or compression. Reinforced masonry is partially grouted or fully grouted.

Running bond: A pattern of masonry where the head joints are staggered between adjacent courses by more than a third of the length of a masonry unit. Also refers to the placement of masonry units such that head joints in successive courses are horizontally offset at least one-quarter the unit length.

Shear wall: A wall that resists lateral forces applied parallel with its plane. Also known as an in-plane wall.

Solid masonry unit: A masonry unit whose net cross-sectional area in every plane parallel to the bearing surface is 75% or more of the gross cross-sectional area in the same plane.

Solid wall or solid infill panel: A wall or infill panel with openings not exceeding 5% of the wall surface area. The maximum length or height of an opening in a solid wall must not exceed 10% of the wall width or story height. Openings in a solid wall or infill panel must be located within the middle 50% of a wall length and story height, and must not be contiguous with adjacent openings.

Stack bond: In contrast to running bond, usually a placement of units such that the head joints in successive courses are aligned vertically.

Transverse wall: A wall that is oriented transverse to the in-plane shear walls, and resists lateral forces applied normal to its plane. Also known as an out-of-plane wall.

Unreinforced masonry (URM) wall: A masonry wall containing less than the minimum amounts of reinforcement as defined for masonry (RM) walls. An unreinforced wall is assumed to resist gravity and lateral loads solely through resistance of the masonry materials.

Wythe: A continuous vertical section of a wall, one masonry unit in thickness.

7.9 Symbols

A_b Sum of net mortared area of bed joints above and below test unit, in.^{2}

A_{es} Area of equivalent strut for masonry infill, in.^{2}

A_n Area of net mortared/grouted section of wall or pier, in.^{2}

A_{ni} Area of net mortared/grouted section of masonry infill, in.^{2}

A_s Area of reinforcement, in.^{2}

E_{fe} Expected elastic modulus of frame material, psi

E_{me} Elastic modulus of masonry in compression as determined per Section 7.3.2.2, psi

E_{se} Expected elastic modulus of reinforcing steel per Section 7.3.2.6, psi

G_{me} Shear modulus of masonry as determined per Section 7.3.2.5, psi

I Moment of inertia of section, in.^{4}

I_{col} Moment of inertia of column section, in.^{4}

I_f Moment of inertia of smallest frame member confining infill panel, in.^{4}

L Length of wall, in.

L_{inf} Length of infill panel, in.

M Moment on masonry section, in.-lb

M/V Ratio of expected moment to shear acting on wall or pier

P_c Lower bound of vertical compressive strength for wall or pier, lb

P_{CE} Expected vertical axial compressive force for load combinations in Equations 3-14 and 3-15, lb

P_{CL} Lower bound of vertical compressive force for load combination of Equation 3-3, lb

Q_{CE} Expected strength of a component or element at the deformation level under consideration in a deformation-controlled action

Q_{CL} Lower bound estimate of the strength of a component or element at the deformation level under consideration for a force-controlled action

Q_E Unreduced earthquake demand forces used in Equation 3-14

Q_G Gravity force acting on component as defined in Section 3.2.8

V Shear on masonry section, lb

V_{bjs} Expected shear strength of wall or pier based on bed-joint shear stress, lb

V_{CL} Lower bound shear capacity, lb

V_{dt} Lower bound of shear strength based on diagonal tension for wall or pier, lb

V_{fre} Expected story shear strength of bare frame, lb

V_{ine} Expected shear strength of infill panel, lb

V_{mL} Lower bound shear strength provided by masonry, lb

V_r Expected shear strength of wall or pier based on rocking, lb

V_{sL} Lower bound shear strength provided by shear reinforcement, lb

V_{tc} Lower bound of shear strength based on toe compressive stress for wall or pier, lb

V_{test} Measured force at first movement of a masonry unit with in-place shear test, lb

a Equivalent width of infill strut, in.

c Fraction of strength loss for secondary elements as defined in Figure 7-1

d Wall, pier, or infill inelastic drift percentage as defined in Figure 7-1

d_v Length of component in direction of shear force, in.

e Wall, pier, or infill inelastic drift percentage as defined in Figure 7-1

f_a Lower bound of vertical compressive stress, psi

f_{ae} Expected vertical compressive stress, psi

f_{me} Expected compressive strength of masonry as determined in Section 7.3.2.1, psi

f_{te} Expected masonry flexural tensile strength as determined in Section 7.3.2.3, psi

f_{vie} Expected shear strength of masonry infill, psi

f_y Lower bound of yield strength of reinforcing steel, psi

f_{ye} Expected yield strength of reinforcing steel as determined in Section 7.3.2.6, psi

f'_{dt} Lower bound of masonry diagonal tension strength, psi

f'_m Lower bound of masonry compressive strength, psi

f'_t Lower bound masonry tensile strength, psi

h Height of a column, pilaster, or wall, in.

h_{col} Height of column between beam centerlines, in.

h_{eff} Height to resultant of lateral force for wall or pier, in.

h_{inf} Height of infill panel, in.

k Lateral stiffness of wall or pier, lb/in.

l_{beff} Assumed distance to infill strut reaction point for beams as shown in Figure C7-5, in.

l_{ceff} Assumed distance to infill strut reaction point for columns as shown in Figure C7-4, in.

p_{D+L} Expected gravity stress at test location, psi

q_{ine} Expected transverse strength of an infill panel, psf

r_{inf} Diagonal length of infill panel, in.

t Least thickness of wall or pier, in.

t_{inf} Thickness of infill panel, in.

v_{me} Expected masonry shear strength as determined by Equation 7-1, psi

v_{te} Average bed-joint shear strength, psi

v_{to} Bed-joint shear stress from single test, psi

Δ_{inf} Deflection of infill panel at mid-length when subjected to transverse loads, in.

α Factor equal to 0.5 for fixed-free cantilevered shear wall, or 1.0 for fixed-fixed pier

β Ratio of expected frame strength to expected infill strength

θ Angle between infill diagonal and horizontal axis, $\tan\theta = L_{inf}/h_{inf}$, radians

θ_b Angle between lower edge of compressive strut and beam as shown in Figure C7-5, radians

θ_c Angle between lower edge of compressive strut and beam as shown in Figure C7-4, radians

κ A reliability coefficient used to reduce component strength values for existing components based on the quality of knowledge about the components' properties (see Section 2.7.2)

λ_1 Coefficient used to determine equivalent width of infill strut

λ_2 Infill slenderness factor

ρ_g Ratio of area of total wall or pier reinforcement to area of gross section

7.10 References

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