# C7. Masonry (Systematic Rehabilitation)

## C7.1 Scope

The scope of Chapter 7 is limited to masonry elements that are considered to resist lateral seismic forces as structural members. The chapter includes walls and infill panels subjected to in-plane and out-of-plane forces. Material given is intended to be used directly with the Analysis Procedures prescribed in Chapter 3. All other masonry elements are addressed in Chapters 4 and 11.

## C7.2 Historical Perspective

### C7.2.1 General

Masonry is the oldest of all construction materials, dating back more than eight millennia to cultures around the globe. Early masonries consisted of stone units with no mortar. The structural action in this form of masonry is much different than that of modern-day clay-unit and concrete masonry, which is found in nearly all existing masonry buildings in the United States, with the exception of some historic buildings that predate the 1850s.

Most masonry buildings in the United States constructed before the 20th century consisted of unreinforced clay-unit masonry. Wythes of brick were commonly tied with brick headers spaced at every sixth or seventh course. Because no other construction material was used for the walls, these building systems represented the first introduction to engineered masonry construction, although seismic considerations were often neglected in the design. Early mortars consisted of no more than lime and sand, which made the shear and tensile strength of the masonry quite weak. In the same era, clay-unit masonry was also used extensively for infills and cladding on steel frame buildings. Though the structural properties of the masonry were ignored in favor of the strong but flexible steel frames, considerable lateral-force resistance was provided by the stiff but brittle masonry, as evidenced by substantial cracking when subjected to earthquake motions.

Following the 1933 Long Beach earthquake, unreinforced masonry (URM) was banned in California, giving rise to reinforced masonry (RM) construction. Today, buildings approaching thirty stories are constructed with stiff, strong, and ductile RM walls designed with limit states concepts. Both hollow-clay and concrete block construction have competed with reinforced concrete and structural steel for the design of commercial, residential, and industrial buildings. In addition, clay-unit masonry remains as the most prevalent material for cladding and veneer on all types of buildings.

In this section, a short treatise on the history of masonry materials is presented to educate the user of these guidelines. Historical summaries are given for:

- clay units
- structural clay tile
- concrete masonry units
- mortar
- reinforced masonry

### C7.2.2 Clay Units

Although brick was one of the first products that people manufactured from clay, the era of modern brick began only when extrusion machines were developed. A few bricks were being made by machine in 1833, but the percentage was small until 1870. With the invention of the extrusion or stiff-mud brick-making machine, some manufacturers produced brick containing holes or “cores” running parallel to either the length or the height dimension of the unit. These cores were introduced as an aid to uniform drying of the clay and as a means of reducing the weight of the unit.

The General Assembly of New Jersey passed a law in 1883 to establish brick dimensions at 9-1/2" x 4-1/2" x 2-3/4". In 1889, in the District of Columbia, the ordinance of October 31, 1820 was still being enforced, which fixed a minimal size of brick at 9-1/4" x 4-5/8" x 2-1/4".

In 1929, a report prepared by McBurney and Logwell summarized that 92% of the brick produced in the United States had flat-wise compressive strengths averaging 7,246 psi for both hard and salmon brick. From the distribution data given, approximately 6% of the production classified as 1,250 to 2,500 psi, 20% as 2,225 to 4,500 psi, and 74% as over 4,500 psi. Approximately 40% of the production was 8,000 psi or over in compressive strength.

Solid brick is now defined as a small building unit, solid or cored not in excess of 25%, commonly in the form of a rectangular prism, formed from inorganic, nonmetallic substances, and hardened in its finished shape by heat or chemical action. Brick is also available in larger units with cell or core areas up to 60% of the cross section. Such units are typically used for placement of both vertical and horizontal reinforcement. The term “brick,” when used without a qualifying adjective, is understood to mean such a unit or a collection of such units made from clay or shale hardened by heat.

### C7.2.3 Structural Clay Tile

Structural clay tile is a machine-made product first produced in the United States in New Jersey in 1875. Structural clay tiles are characterized by the fact that they are hollow units with parallel cells (hollow spaces). The shape of the unit is controlled by the die through which the clay column is extruded. The ease with which different designs could be produced led to the development of a wide variety of sizes and patterns.

In 1903, the National Fireproofing Corporation of Pittsburgh published a handbook and catalog by Henry L. Hinton, illustrating the products of the company and presenting data for use in the design of segmental and flat arch floors. This catalog is of historical interest, particularly because of the large number of unit designs illustrated. Hundreds of different shapes are shown for use in the construction of tile floor arches, partitions, and walls, and for fireproofing columns, beams, and girders.

Structural clay tile was used extensively during World War I. With lumber in critically short supply, hollow-clay tile was largely relied upon for all types of buildings. Brick and tile were used for the construction of mobilization structures, war housing, defense plants, air fields, and buildings at army and navy bases.

In 1950, structural clay tile was classified under the following types: Structural Clay Load-Bearing Wall Tile, Structural Clay Non-Load-Bearing Tile (partition, furring, and fireproofing), Structural Clay Floor Tile, Structural Clay Facing Tile, and Structural Glazed Facing Tile.

### C7.2.4 Concrete Masonry Units

The earliest specification for hollow concrete block was proposed by the National Association of Cement Users in January 1908. The NACU was organized in 1904 and continued under that name until 1913, when it became known as the American Concrete Institute (ACI).

In 1905, the United States government adopted concrete block for its hospitals, warehouses, and barracks in the Panama Canal Zone and the Philippine Islands.

The 1908 specification called for the block in bearing walls to have an average strength of 1000 psi at 28 days with a minimum of 700 psi. Air space was limited to 33% and absorption was to average not more than 15% (with no single block to exceed 22%). Absorption was to be measured on a block placed in a pan of water at least 2" deep. Fine aggregate had to pass a 1/4" mesh sieve; stone or clean-screened gravel was to go through a 3/4" sieve and be refused on a 1/4" sieve. A 1-3-4 semi-wet mix was recommended for exposed bearing walls, and a 1-3-5 mix for a wet cast block. Portland cement mortar was recommended. Transverse, compressive, and absorption tests were required, along with freezing and fire tests when necessary, and the modulus of rupture at 28 days was to average 150. Any expense attending such tests was to be met by the manufacturer of the block.

This first standard specification was adopted in 1910. Two years later, the practice for curing—which until that time had consisted of sprinkling with water for seven days—was revised slightly by the addition of a new method, the use of steam from 100 to 130°C for 48 hours with a subsequent storing of eight days. This recommended practice was the first mention of high-pressure steam curing in block specifications.

In 1916, the absorption rate was changed to 10% at the end of 48 hours. In 1922 came the first specification for a non-load-bearing unit, with a requirement of 300 psi. That same year, the following strengths were suggested: 250, 500, 700, and 1200 psi for non-load-bearing, light-load-bearing, medium-load-bearing, and heavy-load-bearing walls, respectively. The ACI accepted these values as tentative in 1923. The absorption time, however, was shortened from 48 to 24 hours. A similar table, with the elimination of the light-load-bearing unit, was accepted as tentative in 1924, and adopted the following year.

By 1928, more than 80 city building codes had been revised to eliminate practically all of the legal obstacles to the increased use of concrete block. Public works construction by state and local governments had declined steadily until by 1933 it had virtually ceased. In 1933, several government agencies were set up to purchase concrete block. In July 1935, the National Industrial Recovery Act was invalidated by the U.S. Supreme Court, but it had by then performed a valuable service for the concrete block industry. Although business activity in the 1930s was in a constantly deepening trough of despair, lifted only by public building programs, the decade was surprisingly productive in a good many technological areas for the concrete block industry.

### C7.2.5 Mortar

The common variety of mortar was made of lime, sand, and water. Details of its preparation varied according to regional customs and individual preferences, but most of these details were well known throughout Europe and America. Sand was added to lime for economy, to prevent shrinkage, and in such quantity that the lime would fill the interstices. If an excess of sand was used, the bond was poor. If too little sand was used, the mortar would shrink and crack.

In ordinary sands, the spaces were from 39% to 40% of the total volume, and in such, 1.0 volume of cementitious paste (cement plus lime) would fill voids of 2.5 volumes of sand. In practice, 1.25 to 2.0 volumes of sand to 1.0 of paste was used. Thus, “pure” lime mortar meant three to five volumes of sand to one measured volume of lime. This gave a plastic mortar that did not crack.

Until about 1890, the standard mortar used for masonry in the United States was a mixture of sand and pure lime (i.e., hydraulic lime) or lime-pozzolon-sand. Massachusetts Hall (1730) at Harvard University and Independence Hall (1734) in Philadelphia were built with lime mortars that were also known as “fact” mortars. These low-strength mortars gave masonry a low modulus of elasticity and, therefore, an ability to absorb considerable strain without inducing high stress. Accordingly, the tendency to crack was reduced, and when cracks did appear, masonry of high lime-content mortar was to a great extent capable of chemical reconstitution, i.e., “autogenous healing.”

After 1819, all masonry used in the construction of the Erie Canal was laid in natural cement mortar. Various sources afford different information about the mortar mix; apparently one part of sand was mixed with two parts of cement. The general practice in New York state in about 1840 was to mix two or three parts of sand to one of cement.

For natural cements, the proportion of sand to cement by measurement usually did not exceed three to one, and for piers and first-class work a ratio of two to one was used. Portland cement mortar commonly contained four parts of sand to one of cement for ordinary mortar, and three to one for first-class mortar. For work under water, not more than two parts of sand to one of cement were used. When cheaper mortars than these were desired, it was considered better to add lime to the mortar than more sand. Cement mortars were introduced about 1880. Joints of cement mortar were strong and unyielding because of the cement; they were appropriate for bonding to modern bricks and concrete blocks.

### C7.2.6 Reinforced Masonry

Reinforced brick masonry was first used by Marc Isambard Brunel in 1825, in the building of the Thames Tunnel in England (Plummer and Blume, 1953). Reinforced brick masonry was used by many builders during that century; however, these builders were individuals who had a feel for materials and built their structures based upon their experience, more as an art than from a rational design. Prior to 1880, a few attempts were made to develop design formulas. However, the performance of composite steel and masonry flexural members was not clearly understood at that time, and many investigators have attributed the strength of the construction primarily to the adhesive properties of the masonry. In fact, most of the early tests were designed to demonstrate the increased strength obtainable through the use of a new Portland cement in mortar, instead of the hydraulic limes and natural cements formerly used.

In the United States, Hugo Filippi, C.E. built and tested reinforced brick masonry beams in 1913. Later in 1919, L.J. Mensch, C.E. of Chicago also tested reinforced brick beams in which the reinforcement was placed in a bed of mortar below the brick masonry. However, the data from these tests and others were never published and there was little, if any, exchange of information among those interested in the subject.

In 1923, the Public Works Department of the Government of India published Technical Paper #38, a comprehensive report of extensive tests of reinforced brick masonry structures extending over a period of about two years. A total of 282 specimens were tested, including reinforced brick masonry slabs of various thicknesses, reinforced brick beams, both reinforced and unreinforced columns, and reinforced brick arches. These tests appeared to be the first organized research on reinforced brick masonry; the data provided answers to many questions regarding this type of construction. This research may therefore be considered as marking the initial stage of the modern development of reinforced brick masonry.

The idea of using cement-sand grout instead of bonding brick headers to bind brick wythes or tiers together, and inserting reinforcing steel in the grout space for tensile and shearing resistance, was developed for practical and sound engineering use in southern California beginning about 1935. Since then, thousands of tests have been conducted on full-size beams, slabs, and walls, from which sound engineering design criteria have been established and incorporated into building codes throughout the United States.

## C7.3 Material Properties and Condition Assessment

### C7.3.1 General

The term “masonry” is used to define the composite of units, mortar, and possibly grout and/or reinforcement. Whereas there are specifications to control the manufacture of each of the constituent materials, the most basic engineering properties to consider for analysis of a building system are those representing the composite. Thus, permissible values are given in this section for compressive strength and elastic modulus of the masonry assemblage, flexural tensile strength at the unit-mortar interface, and shear strength and shear modulus of vertical components such as piers, panels, and walls. These mechanical properties will be relied on for estimating stiffness and strength of masonry wall and infill components.

### C7.3.2 Properties of In-Place Materials

#### C7.3.2.1 Masonry Compressive Strength

Three options are given for measuring expected masonry compressive strength. The first two methods rely on testing of either extracted or rebuilt masonry prisms in a laboratory. The third method measures strength in situ by inserting a pair of flat jacks in an existing masonry wall.

For the first method, sample test prisms are extracted from a masonry component and transported to a laboratory. The test prisms are subjected to vertical compressive stress until the peak strength is reached. The prism height should be at least twice its thickness, contain at least two bed joints, and be a minimum of 15 inches high. The advantage of this method is that an actual prism can be tested under controlled laboratory conditions. In addition, strains can be monitored to infer the elastic modulus (see Section C7.3.2.2). The disadvantages are that the compressive strength might be reduced during extraction, and the number of test specimens is limited because of the cost of both the extraction and the repair of the wall.

The second method requires test prisms to be fabricated from actual masonry units that are extracted from an existing masonry component. A chemical analysis of the mortar is required so that mix proportions can be simulated, and the mortar can be recreated. The advantage of this method is the same as for the first method. The disadvantage is that long-term creep, moisture, and temperature effects cannot be simulated.

The third method consists of cutting slots in two mortar bed joints, four to six courses apart, so that thin, hydraulic flat jacks can be inserted and pressurized. The portion of the masonry between the jacks is subjected to a state of vertical compressive stress. The jacks are stressed until the strength of the masonry is reached. For masonry that is relatively weak, softening can be observed by a reduction in slope of the stress-strain curve, and compressive strength can be inferred. The advantage of this method is that it is nondestructive and the strength is measured in situ. In addition, the test can be done in concert with other tests done to measure elastic modulus and in situ compressive stress. The disadvantage is that typical flat jacks may not be able to reach the high pressures needed to approach the ultimate strength of the masonry in compression.

As an alternative to the test methods given in the *Guidelines*, the expected masonry compressive strength may be deduced from a nominal value prescribed by the Masonry Standards Joint Committee specification for new construction (MSJC, 1995) knowing the unit strength and mortar type (Specification Table 1 for clay-unit masonry and Table 2 for concrete masonry). Tests of extracted masonry units may be done to ascertain the unit strength, or conservative estimates of unit strength can be assumed for use with the MSJC tables. Likewise, mortar type can be evaluated experimentally or assumed. The MSJC table values are based on data from masonry constructed after the 1950s and are only applicable to this period of construction. Many of the earlier mortars were lime-based rather than cement-based as assumed with these table values. Furthermore, earlier mortars were classified with a different nomenclature than given in these tables, making direct relations difficult. Therefore, the unit-strength procedure using the MSJC tables should only be used for masonry constructed after 1960. Expected masonry strength should be determined by multiplying Table 1 values by a factor of 2.0 or Table 2 values by a factor of 1.5. These approximate factors are based on estimated ratios between expected and lower bound compressive strengths, as well as on correction factors for clay brick and concrete block prisms.

Default values of compressive strength are set at very low stresses to reflect an absolute lower bound. Masonry in poor condition is given a strength equal to one-third that for masonry in good condition, to reflect the influence of mortar deterioration and unit cracking on compressive strength.

### C7.3.2.2 Masonry Elastic Modulus in Compression

The elastic modulus of masonry in compression can be measured by one of two methods. Each method measures vertical strain between two gage points to infer strain, and thus elastic modulus. The first method consists of extracting a test prism from an existing wall; the second method utilizes a pair of flat jacks to subject an in situ portion of masonry to vertical compressive stress.

The extracted prism method is essentially the same as for the compressive strength test, with the difference that dial gages or electronic displacement transducers are fixed on the test prism to measure strain between two gage points.

The flat-jack method is done in the same way as for the compressive strength test, with the difference that the jacks are pressured to less than half the masonry strength. Vertical contractions of the compression field between the two jacks are measured with a mechanical dial gauge or electronic displacement transducer. Strain is then determined by dividing measured distortion by the length between gauges. Using correction factors for shape and stiffness of flat jacks, vertical compressive stress is inferred from measured hydraulic pressure. The elastic modulus, $E_{me}$, is calculated as the slope of the stress-strain curve between 5% and 33% of the estimated masonry ultimate compressive strength.

The flat-jack method has been shown to be accurate within 10%, based on correlations between test values and measured elastic moduli of test prisms (Epperson and Abrams, 1989; Noland et al., 1987). A case study using this method is presented by Kariotis and Ngheim (1995). An available standard is the *Standard Test Method for In-Situ Elastic Modulus within Solid Unit Masonry Estimated Using Flat Jack Measurements*, ASTM C 1197.

Default values of elastic modulus shall be based on a coefficient of 550 times the expected masonry compressive strength. This coefficient is set lower than previous values given in the *Uniform Building Code* to compensate for larger values of expected strength.

### C7.3.2.3 Masonry Flexural Tensile Strength

Although the flexural tensile strength of older brick masonry walls constructed with lime mortars may often be neglected, the tensile strength of newer concrete and clay-unit masonry walls can result in appreciable flexural strengths. Therefore, guidelines for measuring flexural tensile strength in situ or from extracted specimens are given in this section.

Masonry flexural tensile strength can be measured using a device known as a bond wrench, which clamps onto the top course of a test specimen and applies a weak-axis bending moment until the top masonry unit snaps off. Flexural tensile stress is inferred by dividing the moment capacity by the section modulus of the wall section. The test can be done on test specimens extracted from an existing wall, or in situ on a portion of masonry that has been isolated by cutting vertical slots on either side of the test portion. Alternatively, flexural tension stress can be measured by bending extracted portions of a masonry wall across a simply supported span.

For the field test, two adjacent units of a running bond pattern are removed so that a clamp may be inserted. Single masonry units above and below the removed units are subjected to an out-of-plane moment using a calibrated torque wrench. Mortar head joints on either sides of the tested units are removed to isolate the test units. The laboratory test is done in much the same manner on specimens that are cut from a wall. Test prisms should be at least two units in height, and one unit long, or a minimum of four inches. Both methods involve substantial repair of the existing wall. An available standard for the laboratory method is *Standard Test Methods for Masonry Bond Wrench Testing*, ASTM C 1072. No standards exist on the field bond wrench test; however, this ASTM standard should suffice.

The third method consists of extracting sample panels or prisms from an existing masonry wall, and subjecting them to minor-axis bending with either a third-point loading or a uniform load distribution with an airbag. Flexural tensile strength is determined by dividing the maximum applied moment by the section modulus of the masonry section. *Standard Test Methods for Masonry Flexural Tension Stress*, ASTM E 518, is available; however, ASTM does not recommend this method for determination of design stresses.

For all three of these methods, the bonding of the test unit to the mortar is sensitive to any disturbances that are incurred during specimen removal. The confidence level can be low because the scatter of data for flexural bond strength can be high, and the number of test samples is limited because of cost and the disturbance concerns.

These test methods are intended for out-of-plane strength of unreinforced masonry walls. For in-plane bending, flexural stress gradients across the section width are much lower than for out-of-plane bending. Thus, data from tests described in this section should not ideally be used for in-plane bending. However, in lieu of data on in-plane tensile strength, out-of-plane strength values may be substituted.

Default values for flexural tensile strength are set low even for masonry in good condition, because of the dependence of the unit-mortar bonding on the tensile strength. This bonding can be highly variable, depending on the relative absorption of the unit and the water retentivity of the mortar, the presence and type of cement used in the mortar, the previous loading history, and the condition of the mortar. For masonry in poor condition, a zero value of tensile strength is prescribed.

### C7.3.2.4 Masonry Shear Strength

Expected shear strength of URM components can be inferred from in situ measurements of bed-joint shear strength using the in-place shear test. The nondestructive test measures the in situ shear strength between a clay masonry unit and the mortar bed joints above and below the unit. A small hydraulic jack is placed in a void left by removal of a masonry unit immediately adjacent to the test unit. The head joint on the opposite face of the test unit is removed to isolate the test unit so that it may be displaced horizontally when pushed.

A horizontal force is applied to the test unit until it starts to slide. Shear strength is then inferred as the measured force divided by the area of the bed joints above and below the masonry unit. The estimated vertical compressive stress at the test location is subtracted from this value to give the bed joint shear stress, $v_{to}$ (Equation 7-2), assuming a coefficient of friction equal to 1.0. Because expected values of wall shear strength are to be used, the 50th percentile value, $v_t$, is used as the index value.

The method is limited to tests of the face wythe. When the test unit is pushed, resistance is provided across not only the bed-joint shear planes, but also the collar-joint shear plane. Because seismic shear is not transferred across the collar joint in a multiwythe masonry wall, the estimated shear resistance of the collar joint must be deducted from the test values. This is done by multiplying the $v_{te}$ term by 0.75 in Equation 7-1, which is the ratio of the areas of the top and bottom bed joints to the sum of the areas of the bed and collar joints for a typical clay unit. If it is known that the collar joint is not present, or is in very poor condition, the 0.75 factor may be waived.

The effect of friction at the particular location of the masonry element being evaluated is included by increasing the bed-joint shear capacity by the addition of the term “P/A” in Equation 7-1. The sum is then multiplied by a reduction factor equal to 0.75, and divided by 1.5 to convert it to an average stress for use with walls of a rectangular cross section.

The in-place shear test was developed solely for solid clay-unit masonry. However, the test method has been used for single-wythe hollow concrete block masonry. As for the conventional method with brick masonry, a single unit is removed adjacent to a test unit as well as the opposite mortar head joint. The maximum horizontal force needed to move the block is divided by the total area of the bed joint mortar above and below the test unit and the total grouted area. The term $v_{to}$ is obtained by subtracting the apparent vertical compressive stress from this ratio as given in Equation 7-2. If the shear capacity of the masonry exceeds that of the loading equipment, the test may be run on one-half the length of a block. In such case, the mortar bed joints along one-half the length of the block are removed.

An alternate in-place shear test method is to simultaneously apply a vertical compressive stress, using hydraulic flat jacks placed in the bed joints above and below the test brick, while shearing the test brick. In-place shear tests are done at various levels of vertical compressive stress so that values of cohesion and frictional coefficients can be inferred.

The available standard *In-Place Masonry Shear Tests* (UBC Standard 21-6), is referenced in the 1994 *Uniform Code for Building Conservation* (ICBO, 1994), Appendix Chapter 1, Sections A106(c)3 and A107(b).

Default values for shear strength of URM are provided, ranging from 27 psi for good condition to 13 psi for poor condition. If in-place shear tests are done, the upper bound of $v_{me}$ by Equation 7-1 is 37 psi for a zero vertical compressive stress when the 100 psi limit on $v_{te}$ is considered. Thus, a 37% increase in strength is possible if testing is done and the masonry is considered to be in good condition. Default values for shear strength of poor masonry are large relative to values for masonry in good condition (1:2), because frictional shear can be developed even when mortar or units are deteriorated.

Shear strength of reinforced masonry (RM) cannot be expressed in terms of the bed-joint shear stress because of the influence of the vertical and horizontal reinforcement on shear strength. There are no in situ methods for measuring shear strength of existing RM walls. Equations given for shear strength in BSSC (1995) must be relied on. Ideally, the theory of mechanics of materials does not change with age, and the same strength equations should apply for existing or new construction. However, care should be taken to ensure that the condition of the existing masonry components is comparable to that of newly constructed elements. This assessment should include a review of reinforcing details as well as the general condition of the masonry (see Section 7.3.3).

#### C7.3.2.5 Masonry Shear Modulus

Laboratory tests of URM shear walls (Epperson and Abrams, 1989; Abrams and Shah, 1992) have found that the shear modulus of masonry does approach the value of 0.4 times the elastic modulus in compression, as given by the theory of elasticity for isotropic, elastic members. This value is limited to elastic, uncracked behavior of the masonry. After cracking, the shear stiffness is known to reduce substantially as sliding along bed joints develops or as diagonal tension cracks open. Because these nonlinear effects cannot be related to the elastic modulus in compression, the $0.4E_m$ value is only appropriate for uncracked masonry. Shear stiffness of post-cracked masonry can be taken as a fraction of the initial shear stiffness. Test data by Atkinson et al. (1989) provide estimates of shear stiffness based on a frictional mechanism along bed joints.

#### C7.3.2.6 Strength and Modulus of Reinforcing Steel

The expected strength of reinforcing bars can be best determined from tension tests of samples taken from the building. If available, mill test data for the reinforcing steel used in the building may be substituted.

Default values of yield strength are given to be the same as for reinforcing bars in reinforced concrete (see Section 6.3.2.5).

#### C7.3.2.7 Location and Minimum Number of Tests

The required number of tests have been established based on theories of statistical sampling, and past experience.

### C7.3.3 Condition Assessment

The goals of a condition assessment are:

- To examine the physical condition of primary and secondary components and the presence of any degradation
- To verify the presence and configuration of components and their connections, and continuity of load paths between components, elements, and systems
- To review other conditions, such as neighboring party walls and buildings, presence of nonstructural

components, and limitations for rehabilitation, that may influence building performance

- To formulate a basis for selecting a knowledge factor

The physical condition of existing components and elements, and their connections, should be examined for deterioration of masonry units, mortars, grouts, and reinforcement. Deterioration may include environmental effects (e.g., fire damage, chemical attack, freeze/thaw damage) or past/current loading effects (e.g., overload, damage from past earthquakes, cracking). Masonry construction is also susceptible to expansion and contraction due to thermal and moisture conditions.

A condition assessment should examine configuration problems such as discontinuous reinforcement patterns, unequal alignment of components, and inadequate connections between walls and foundation.

The scope of a condition assessment shall include an investigation of primary and secondary structural elements and components. Although masonry veneer is not part of the structural system, the condition and attachment of the veneer should be examined. Substantial damage to masonry veneer has been observed in numerous earthquakes (Klingner, ed., 1994). Rehabilitation measures should be undertaken to mitigate damage to veneer. However, since the veneer is not part of the structural system, such measures will not involve the Systematic Rehabilitation procedures prescribed in Chapter 7. Accessibility constraints may necessitate the use of instruments such as a fiberscope or video probe, to reduce the amount of damage to covering materials and fabrics. The knowledge and insight gained from the condition assessment are invaluable to the understanding of load paths and the ability of components to resist and transfer these loads.

Destructive or nondestructive test methods may be necessary to examine the interior portions of a masonry structural component. Local removal of sheathing or coatings on masonry wall surfaces may need to be done to expose connections between the masonry and adjoining components. The number of such examinations will vary with the complexity and availability of construction drawings.

### C7.3.3.1 Visual Examination

Visual observations are simple and generally inexpensive, and can detect obvious condition states in the masonry materials and quality of construction. Configuration problems can quickly be identified with direct visual inspection. The continuity of load paths can be determined through viewing of components and connection condition. Visual inspection can determine the need for other test methods to quantify the presence and degree of deterioration.

The process of establishing component properties should start with obtaining construction documents. Preliminary review of these documents should be done to identify primary gravity- and lateral-load-carrying elements, systems, components, and connections. In the absence of a complete set of building drawings, a thorough inspection of the building should be done to identify all load-bearing systems, and an as-built set of drawings should be made.

If coverings or other obstructions exist, indirect visual inspection can be done through use of drilled holes and a fiberscope.

### C7.3.3.2 Nondestructive Tests

Four tests are recommended to assess the relative condition of masonry components: ultrasonic pulse velocity, mechanical pulse velocity, impact echo, and radiography. Merits and limitations of each method are described in this section. Further information can be found in Abrams and Matthys (1991).

#### A. Ultrasonic Pulse Velocity

Measurement of the velocity of ultrasonic pulses through a wall can detect variations in the density and modulus of masonry materials as well as the presence of cracks and discontinuities. Transmission times for pulses traveling through a wall (direct method) or between two points on the same side of a wall (indirect method) are measured and used to infer wave velocity.

Test equipment with wave frequencies in the range of 50 kHz has been shown to be appropriate for masonry walls. Use of equipment with higher-frequency waves is not recommended because the short wave length and high attenuation are not consistent with typical dimensions of masonry units.

Test locations should be sufficiently close to identify zones with different properties. Contour maps of direct transmission wave velocities can be constructed to assess the overall homogeneity of a wall elevation. For indirect test data, vertical or horizontal distance can be plotted versus travel time to identify changes in wave velocity (slope of the curve). Abrupt changes in slope will identify locations of cracks or flaws.

Ultrasonic methods are not applicable for masonry of poor quality or low modulus, or with many flaws and cracks. The method is sensitive to surface condition, the coupling material used between the transducer or receiver and the brick, and the pressure applied to the transducer.

The use of ultrasonic pulse velocity methods with masonry walls has been researched extensively (Calvi, 1988; Epperson and Abrams, 1989; Kingsley et al., 1987). A standard for the use of ultrasonic methods for masonry is currently under development in Europe with RILEM Committee 76LUM.

#### B. Mechanical Pulse Velocity

The mechanical pulse velocity test consists of impacting a wall with a hammer blow and measuring the travel time of a sonic wave across a specified gage distance. An impact hammer is equipped with a load cell or accelerometer to detect the time of impact. A distant accelerometer is fixed to a wall to detect the arrival time of the pulse. Wave velocity is determined by dividing the gage length by the travel time. The form and duration of the generated wave can be varied by changing the material on the hammer cap.

The generated pulse has a lower frequency and higher energy content than an ultrasonic pulse, resulting in longer travel distances, and less sensitivity to small variations in masonry properties and minor cracking. The mechanical pulse method should be used in lieu of the ultrasonic pulse method when overall mean properties of a large portion of masonry are of interest.

The use of mechanical pulse velocity measurements for masonry condition assessments has been confirmed through research (Epperson and Abrams, 1989; Kingsley et al., 1987). Although no standard exists for mechanical pulse velocity tests with masonry, a standard for concrete materials does exist, which may be referenced: *Test Method for Pulse Velocity through Concrete (10-150 kHz range)*, ASTM C 597.

#### C. Impact Echo

The impact-echo technique can be useful for nondestructive determination of the location of void areas within grouted reinforced walls (Sansalone and Carino, 1988). Commercial devices are available or systems can be assembled using available electronic components. Since this technique cannot distinguish between a shrinkage crack at the grout-unit interface and a complete void in the grout, drilling of small holes in the bed joint or examination using an optical borescope should be performed to verify the exact condition.

#### D. Radiography

A number of commercial devices exist that can be used to identify the location of reinforcing steel in masonry walls. They are also useful for locating bed-joint reinforcing steel, masonry ties and anchors, and conduits and pipes. The better devices can locate a No. 6 bar at depths up to approximately six inches; however, this means that for a 12-inch-thick concrete masonry wall, a bar located off-center cannot be found when access is limited to only one side of the wall. These devices are not able to locate or determine the length of reinforcing bar splices in walls for most cases. They work best for identifying the location of single isolated bars, and become less useful when congestion of reinforcing bars increases.

### C7.3.3.3 Supplemental Tests

#### A. Surface Hardness

The surface hardness of exterior-wythe masonry can be evaluated using the Schmidt rebound hammer. Research has shown that the technique is sensitive to differences in masonry strength, but cannot by itself be used to determine absolute strength. A Type N hammer (5000 lb.) is recommended for normal-strength masonry, while a Type L hammer (1600 lb.) is recommended for lower-strength masonry. Impacts at the same test location should be continued until consistent readings are obtained, because surface roughness can affect initial readings.

The method is limited to tests of only the surface wythe. Tuckpointing may influence readings and the method is not sensitive to cracks.

Measurement of surface hardness for masonry walls has been studied (Noland et al., 1987).

#### B. Vertical Compressive Stress

In situ vertical compressive stress resisted by the masonry can be measured using a thin hydraulic flat jack that is inserted into a removed mortar bed joint. Pressure in the flat jack is increased until distortions in the brickwork are reduced to the pre-cut condition. Existing vertical compressive stress is inferred from the jack hydraulic pressure, using correction factors for the shape and stiffness of the flat jack.

The method is useful for measurement of gravity load distribution, flexural stresses in out-of-plane walls, and stresses in masonry veneer walls that are compressed by a surrounding concrete frame. The test is limited to only the face wythe of masonry.

Not less than three tests should be done for each section of the building for which it is desired to measure in situ vertical stress. The number and location of tests should be determined based on the building configuration, and the likelihood of overstress conditions.

#### C. Diagonal Compression Test

A square panel of masonry is subjected to a compressive force applied at two opposite corners along a diagonal until the panel cracks. Shear strength is inferred from the measured diagonal compressive force based on a theoretical distribution of shear and normal stress for a homogeneous and elastic continuum. Using the same theory, shear modulus is inferred from measured diagonal compressive stress and strain.

Extrapolation of the test data to actual masonry walls is difficult because the ratio of shear to normal stress is fixed at a constant ratio of 1.0 for the test specimens. Also, the distribution of shear and normal stresses across a bed joint may not be as uniform for a test specimen as for an actual wall. Lastly, any redistribution of stresses after the first cracking will not be represented with the theoretical stress distributions. Thus, the test data cannot be useful to predict nonlinear behavior.

If the size of the masonry units relative to the panel dimension is large, masonry properties will be not continuous, but discrete. Test panels should be a minimum of four feet square. The high cost and disruption of extracting a number of panels this size may be impractical.

A standard is available, titled *Standard Test Method for Masonry Diagonal Compression*, ASTM E 519.

#### D. Large-Scale Load Tests

Large-scale destructive tests may be done on portions of a masonry component or element to (1) increase the confidence level on overall structural properties, (2) obtain performance data on archaic building materials and construction materials, (3) quantify effects of complex edge and boundary conditions around openings and two-way spanning, and (4) verify or calibrate analytical models. Large-scale load tests do not necessarily have to be run to the ultimate limit state. They may have value for simply demonstrating structural integrity up to some specific Performance Level.

Out-of-plane strength and behavior of masonry walls can be determined with air-bag tests. Behavior of test panels incorporating connections and edge details can be determined from such a test, in addition to flexural and arching properties of a solid or perforated wall.

Strength and deformation capacity under in-plane lateral forces can be determined by loading an individual portion of wall that is cut free of the surrounding masonry. Loading actuators are reacted against adjacent and stronger portions of masonry. Such testing is particularly useful when the wall is composed of different materials that cannot be evaluated by testing an individual unit of an individual wythe.

Visual and nondestructive surveys should be used to identify locations for test samples.

Standards for laboratory test methods are published by ASTM. Procedures for removal and transportation of masonry samples are given in *Evaluation of Structural Properties of Masonry in Existing Buildings*, NBS Building Science Series 62, U.S. Department of Commerce.

Large-scale tests are expensive and limited to a single or few samples. They may result in considerable local damage and may require substantial reconstruction near the sample location. Test data must be extrapolated to the remainder of the system based on a low confidence level.

## C7.3.4 Knowledge ($\kappa$) Factor

The level of knowledge of a particular masonry structure may conform to either a minimum level or an enhanced comprehensive level. As noted in Section 2.7.2, knowledge factors, $\kappa$, are assigned equal to 0.75 and 1.00 for these two levels. The Linear Static Procedure (LSP) of Chapter 3 may be used with either knowledge level, but the Nonlinear Static Procedure (NSP) is limited to a $\kappa$ factor equal to 1.0.

The basic distinction between the two levels of knowledge is whether or not in situ tests of masonry materials are done. For the minimum level, a visual examination of the structure is required per Section 7.3.3.1; however, in-place testing is not necessary. Thus, the LSP may be used with the default values of material strengths as specified in Section 7.3.2. For the comprehensive level of knowledge, some in situ material testing is required in addition to the nondestructive testing for condition assessment noted in Section 7.3.3.2. These tests include determination of masonry compressive strengths using one of the methods prescribed in Section 7.3.2.1 for both unreinforced and reinforced masonry. For unreinforced masonry only, in-place shear strength tests must be done in accordance with Section 7.3.2.4. For reinforced masonry only, tensile strengths of reinforcing bars must be determined in accordance with Section 7.3.2.6.

Even for the comprehensive level of knowledge, in situ tests of masonry flexural tensile strength or elastic modulus are not required. This is because tensile strength should be quite low and somewhat similar to the default values as given in Section 7.3.2.3. Similarly, test data for elastic modulus can have a large scatter and not differ from the approximate value given in Section 7.3.2.2 (550 times the masonry expected compressive strength).

## C7.4 Engineering Properties of Masonry Walls

Masonry building systems are composed largely of walls. Masonry walls may be divided between structural walls—such as bearing or shear walls—and nonstructural walls, such as partition walls, cladding, veneer, infills, and parapets. Engineering properties given in Section 7.4 apply only to structural walls.

Masonry bearing walls support floor and roof gravity loads, and may or may not be shear walls. Conversely, masonry shear walls resist lateral seismic forces, and may or may not be bearing walls. If a wall is part of the lateral-force-resisting system, it is considered as a primary element. If the wall supports only gravity loads and must remain stable under lateral sway, it is considered as a secondary element. All other masonry walls are excluded from Section 7.4.

### C7.4.1 Types of Masonry Walls

Structural masonry walls are classified into three fundamental types: existing, new, and enhanced. Guidelines for determining structural properties of masonry walls reference current standards, which are different for existing and new walls. In addition, the *Guidelines* provide specific recommendations on minimum requirements for enhancement of existing walls so that their structural properties may be considered the same as those of new or existing elements or components.

Rehabilitated buildings typically consist of lateral-force-resisting systems that comprise a combination of different materials. An existing unreinforced masonry building might be strengthened by adding braced steel frames, or conversely, a new reinforced masonry wall might be added to stiffen a flexible steel frame. Existing masonry walls might be enhanced with shotcrete or surface coatings, reinforced or prestressed cores, grout injections, or repointing, or by changing the size of openings. The engineering properties given in Section 7.4 are applicable to building systems with existing, new, or enhanced masonry walls that combine to rehabilitate a building system.

Stiffness assumptions, strength criteria, and acceptable deflections for various limit states as described in Sections 7.4.2 through 7.4.5 are common for existing, new, or enhanced masonry walls. Principles of mechanics are the same despite the age of a masonry wall. Physically, there should be no difference in stiffness assumptions, strength criteria, or inelastic behavior for existing, newly constructed, or enhanced walls. Thus, guidelines on determining engineering properties for each of the three fundamental wall types are expressed in common in these sections.

In Sections 7.4.2 through 7.4.5, walls are grouped in terms of how they respond to lateral forces. Unreinforced walls are presented first, followed by reinforced walls, because the behavior of each type of wall is distinctively different. Furthermore, walls subjected to in-plane lateral forces are separated from walls subjected to out-of-plane forces because their stiffnesses, strengths, and acceptable deformations vary widely.

### C7.4.1.1 Existing Masonry Walls

Existing masonry walls will have a significant influence on the lateral strength and drift of a building system. Certain masonry walls may have a brittle character, and partial or complete removal may improve the overall energy dissipation capabilities of a system, and may thus be a viable rehabilitation option. When considering a particular rehabilitation scheme, existing masonry walls, or their extraction, should be included in the structural analysis along with any new masonry walls that may be added.

A thorough condition assessment of existing masonry walls should be made to increase the level of confidence in characterizing structural properties.

### C7.4.1.2 New Masonry Walls

Newly constructed masonry walls can be added to an existing building system for the purpose of strengthening, stiffening, or increasing inelastic deformation and energy dissipation capacity. The design of new masonry walls must follow the *NEHRP Recommended Provisions* (BSSC, 1995). This standard is based on strength design for both unreinforced and reinforced masonry walls. When used in combination with existing walls, no capacity reduction, or $\phi$ factors, should be used.

In zones of high seismicity, new masonry walls must be reinforced with at least the minimum percentages of reinforcement as specified for a reinforced wall in Section 7.8 (BSSC, 1995). In zones of moderate seismicity, masonry walls must have a minimum of trim bars at corners, top and bottom and around all openings per the *NEHRP Recommended Provisions.* Unreinforced walls can be added to an existing building in zones of low seismicity since they are recognized by this standard.

### C7.4.1.3 Enhanced Masonry Walls

Both reinforced and unreinforced walls may be rehabilitated by the various means noted in this section to increase their strength, stiffness, and/or deformation resistance capacity. Enhancement methods are not listed in a priority order, nor are they necessarily the sole methods that can be used.

#### A. Infilled Openings

A common method of stiffening or strengthening an in-plane masonry wall is to fill window or door openings with masonry. This is typically done for unreinforced walls, but may also be applicable to reinforced walls if needed.

Infilling of an existing opening will stiffen and strengthen a perforated shear wall. The restriction of opening length to no more than 40% of the overall wall length was intended to limit the introduction of new masonry, which by this provision may be considered to exhibit behavior equal to that of the original masonry. The percentage was chosen so that the majority of masonry would be original.

#### B. Enlarged Openings

Door and window openings in unreinforced masonry walls may be enlarged to alter the aspect ratio of an adjacent pier. By removing a portion of masonry above or below an opening, the height-to-length aspect ratio of the adjacent piers will be increased to such an extent that rocking behavior may govern their response. Although this approach will weaken a perforated masonry wall, it will also increase its inelastic deformation capacity if a ductile rocking mechanism can be invoked. Furthermore, if the method is used, excessive diagonal tensile stresses can be relieved for a relatively stocky pier, thus lowering its vulnerability to nonductile “X” cracking.

The method is also applicable to infill panels. Increasing the size of an opening will reduce infill strength and stiffness and may relieve a surrounding frame from excessive frame-infill interactive forces.

#### C. Shotcrete

Application of reinforced shotcrete to the surface of a masonry wall is a common method for enhancing both in-plane and out-of-plane strength. The shear area of the wall is increased and the height-to-thickness (h/t) ratio is lowered. Reinforcement embedded in the shotcrete layer substantially improves both the shear and flexural capacities. The method may be used with existing reinforced masonry walls, but has its greatest potential with unreinforced walls.

If shotcrete is used to enhance out-of-plane strength, flexural behavior will be asymmetrical for loading in each direction, since the compression zone will alternate between the shotcrete layer and the masonry.

#### D. Coatings for URM Walls

Surface coatings may be used to enhance the in-plane shear strength of a URM wall. The h/t ratio will be reduced with the coating, which will enhance the strength of the wall in compression and under transverse loads. Coatings may consist of a cement plaster coating with an embedded steel mesh, or a gypsum plaster coating.

Research has been done on the effectiveness of using fiber-reinforced composites (e.g., kevlar, carbon fibers) for strengthening masonry walls; however, long-term durability remains questionable.

#### E. Reinforced Cores for URM Walls

Existing URM walls may be reinforced in the vertical direction by grouting reinforcing bars in cores drilled through the wall height. The method, commonly known as the “center core technique,” has been used predominantly in California for seismic rehabilitation of URM buildings. With adequate anchorage of new vertical reinforcing bars in the drilled cores, a wall may be assumed to act as a reinforced wall in flexure.

The use of epoxy resins to fill cores around reinforcing bars in older, softer masonry materials has resulted in accelerated deterioration due to incompatibility of materials.

#### F. Prestressed Cores for URM Walls

Existing URM walls may be prestressed in the vertical direction with strands or rods embedded at their base in grout and placed in cores drilled through the wall height.

Tendons should be ungrouted. Walls enhanced with unbonded tendons will respond in a nonlinear but elastic (returning to undeformed shape) manner. If tendons are bonded with grout, inelastic straining of the tendon can dissipate substantial seismic energy. However, because of the high strength of most tendon steel (cables or bars), excessive compressive strain may result in premature crushing of the masonry before the tendon can develop post-yield strains. Thus, hysteretic damping and ductile performance will be inhibited.

Losses in prestressing force can be estimated based on the expected shortening of a masonry component due to elastic deformations, creep, and shrinkage effects. Design procedures for estimating losses are given in Curtin et al. (1988). Research results on creep and shrinkage movements of clay-unit masonry can be found in Lenczner (1986).

Unlike the reinforced core technique, the prestressed core technique will improve shear strength as well as flexural strength because of the friction that is developed as a result of the increased vertical compressive stress.

#### G. Grout Injections

The shear strength of existing masonry walls can be enhanced by injecting grout into the interior voids of the wall. For unreinforced brick masonry walls, grout can be injected into possible voids in the collar joint in addition to the head and bed joints. This will also increase the shear and tensile strength between wythes and increase the transverse strength of a multiwythe wall. For hollow-unit masonry, grout can be injected into the open cells.

#### H. Repointing

Repointing is the process of removing deteriorated mortar joints and replacing with new mortar. Repointing can be used to enhance shear or flexural strength of a URM wall.

#### I. Braced Masonry Walls

Steel bracing elements can be provided to reduce the span of a masonry wall bending in the out-of-plane direction.

#### J. Stiffening Elements

Additional structural members can be added to enhance the out-of-plane flexural stiffness and strength of a masonry wall. Such members may be placed in the vertical and/or horizontal direction.

## C7.4.2 URM In-Plane Walls and Piers

Walls resisting lateral forces parallel to their plane are termed “in-plane walls.”

Solid walls deflect as vertical cantilevered flexural elements from the foundation. Tall slender in-plane walls (height larger than length) resist lateral forces primarily with flexural mechanisms. Squat walls (height less than length) resist lateral forces primarily with shear mechanisms.

Perforated walls can be idealized as a system of piers and spandrel beams. If beams are sufficiently stiff in bending, piers can be assumed to be fully restrained against rotation at their top and bottom. If openings in a perforated wall are relatively large, the wall system will deflect as a cantilevered shear element from the foundation. Pier distortions in flexure and shear will result in story drifts with little rotation of the floor level.

The provisions of Section 7.4.2 apply to both cantilevered shear walls and individual pier elements adjacent to window or door openings. The difference in rotational boundary conditions at the top of either walls or piers is accounted for with an $\alpha$ factor that increases the lever arm of the vertical compressive force about the toe for a pier type component.

### C7.4.2.1 Stiffness

#### A. Linear Elastic Stiffness

Force-deflection behavior of unreinforced masonry shear walls is linear-elastic before net flexural tension stresses at the wall heel exceed tensile strengths, or diagonal tension or bed-joint sliding shear stresses exceed shear strengths.

Laboratory tests of solid shear walls have shown that behavior can be depicted at low force levels using conventional principles of mechanics for homogeneous materials. In such cases, the lateral in-plane stiffness of a solid cantilevered shear wall, $\kappa$, can be calculated using Equation C7-1:

$$k = \frac{1}{\dfrac{h_{eff}^3}{3E_mI_g} + \dfrac{h_{eff}}{A_vG_m}} \qquad \text{(C7-1)}$$

where:

$h_{eff}$ = Wall height

$A_v$ = Shear area

$I_g$ = Moment of inertia for the gross section representing uncracked behavior

$E_m$ = Masonry elastic modulus

$G_m$ = Masonry shear modulus

Correspondingly, the lateral in-plane stiffness of a pier between openings with full restraint against rotation at its top and bottom can be calculated using Equation C7-2:

$$k = \frac{1}{\dfrac{h_{eff}^3}{12E_mI_g} + \dfrac{h_{eff}}{A_vG_m}} \qquad \text{(C7-2)}$$

where the variables are the same as for Equation C7-1.

Analytical studies done by Tena-Colunga and Abrams (1992) have shown that linear-elastic models can be used to estimate measured dynamic response of an unreinforced masonry building excited during the 1989 Loma Prieta earthquake.

#### B. Nonlinear Behavior of URM Walls

As the lateral force is increased on a wall or pier component, flexural or shear cracking—or a combination of both—will occur, resulting in deflections that are nonlinear with respect to the applied forces. Nonlinear behavior of URM walls has been shown to be dependent on the length-to-height ($L/h$) aspect ratio and the amount of vertical compressive stress.

Behavior of relatively stocky walls ($L/h$ greater than 1.5) is typically governed by diagonal tension or bed-joint sliding, depending on the level of vertical compression, masonry tensile strength, and bed-joint sliding shear strength. For walls governed by diagonal tension, cracks can develop in either a stair-step pattern through mortar head and bed joints, or a straight diagonal path through masonry units. The former action occurs when the mortar is weak relative to the units; the latter occurs when the converse is true. The stair-stepped pattern is better for inelastic deformation capacity because vertical compressive stress normal to the bed joints will result in the development of frictional forces that will remain active at nearly any amount of lateral deflection. Walls governed by a weaker bed-joint sliding shear strength will deform with either a concentrated deformation at one or a few bed joints, or a distribution deformation across several bed joints, depending on the ratio of the cohesion and the frictional coefficient. The inelastic deformability of this sliding type of deformation is also enhanced by frictional forces that remain nearly constant despite the amount of lateral deflection.

In walls with a moderate aspect ratio ($L/h$ between 1.0 and 1.5), considerable strength increases have been observed after flexural cracks form at the heel of a wall as the resultant vertical compressive force migrates towards the compressive toe. As the effective section decreases with progressive cracking, the wall element softens, gradually generating a nonlinear force-deflection relation. If the shear capacity is not reached, the ultimate limit state for such walls is toe crushing. Flexural tension strength at the wall heel does not limit lateral strength. Results from experiments by Epperson and Abrams (1992) and Abrams and Shah (1992) have revealed these tendencies. An analytical study by Xu and Abrams (1992) investigated lateral strength and deflection of cracked unreinforced masonry walls behaving in this range.

For more slender walls ($L/h$ less than 1.0) loaded with a relatively light amount of vertical compressive force, flexural cracks will develop along a bed joint near the base of the wall. When the lateral force approaches a value of $PL/2h$, the wall will start to rock about its toe, provided that the shear strength will not be reached. A singularity condition will exist momentarily as the compressive stress at the wall toe increases rapidly just before rocking, which will cause, at worst, some slight cracking at the toe. Despite the fact that a bed-joint crack will develop across almost all of the wall base, the wall can still transfer shear because of friction at the wall toe as a result of the vertical compressive force. After rocking commences, the wall can be displaced to very large drifts with no further damage as a result of the rigid-body rotation about its toe. Again, flexural tension strength at the wall heel does not limit lateral strength. Behavior in this range has been observed with experiments by Calvi et al. (1996) and Costley and Abrams (1995).

The same types of action can be depicted for pier components; however, the vertical compressive force will shift towards the compression toe at both the top and bottom of the pier. This restraining action will cause the rocking strength to almost double because of the increase in lever arm distance between the vertical force couple. The use of the $\alpha$ factor in Equation 7-4, which accounts for differences in rocking strengths for cantilevered walls and fixed-fixed piers, is explained in Kingsley (1995).

Upon unloading, wall or pier components subjected to rocking actions will resume their original position as a result of the restoring nature of the vertical compressive force. For components subject to bed-joint sliding, the slope of the unloading portion of the force-deflection relation will be steep and will continue after the sense of the deflection is reversed. Unlike a reinforced concrete or masonry beam, the hysteresis loop will not be pinched. Thus, the area enclosed by the loop can be large.

#### C. Lateral Stiffness with Linear Procedures

The linear procedures of Section 3.3 are based on unreduced lateral forces for determination of component actions. If the component is deformation-controlled, these unreduced forces, $Q_{UD}$, are compared with expected component strengths, $Q_{CE}$, multiplied by $m$ factors representing different ductilities. Because the unreduced forces are fictitious, they cannot be used to assess the expected amount of cracking in any component. Thus, reductions in stiffness cannot be estimated because actual force levels are not known. Therefore, only initial, uncracked linear stiffnesses can be used with the equivalent linear procedures. Any nonlinear action is accounted for by applying the $m$ factor to expected strengths.

Much like that of a reinforced concrete beam past yield, the tangent stiffness of a rocking wall or pier is quite small relative to its uncracked stiffness before rocking. For modeling the distribution of story shear to individual piers, the linear stiffness is used rather than the tangent rocking stiffness, which is analogous to the procedure used for strength design of concrete structures. Again, the initial stiffness is used to estimate the elastic demand forces, which are then related to expected strengths by introducing the $m$ factor. Thus, individual pier forces are not distributed in accordance with rocking strengths—as is done with FEMA 178 (BSSC, 1992a) or UCBC procedures—but with respect to relative elastic stiffnesses.

### C7.4.2.2 Strength Acceptance Criteria

As noted in Section C7.4.2.1B, lateral strength of unreinforced in-plane masonry walls or piers is limited by diagonal tension, bed-joint sliding, toe crushing, or rocking. Net flexural tension stress is not a limit for strength, because post-cracked behavior is assumed for the nonlinear range of response.

Rocking and bed-joint sliding are classified as deformation-controlled actions because lateral deflections of walls and piers can become quite large as strengths remain close to constant. Diagonal tension and toe crushing are classified as force-controlled actions because they occur when a certain stress is reached, and can cause sudden and substantial strength deterioration. Stair-stepped diagonal cracking can also be considered as a deformation-controlled action because frictional forces along bed joints are conserved with vertical compressive forces. However, diagonal tension must be classified as a force-controlled action unless stair-stepped cracking can be distinguished from diagonal cracking through units.

**A. Expected Lateral Strength of Walls and Piers**

Expected bed-joint sliding shear strength is determined using Equation 7-3. The expected bed-joint shear strength from in-place shear tests is multiplied by the full area of the mortar and/or grout. Although no shear stress can be developed across flexural bed-joint cracks, the increased compressive stress resisted by the opposite wall or pier edge should compensate for this reduction. For the case of a rocking pier, nearly all of the bed joint may be open at the base and top to accept the component's rotation, yet shear is still transferred at the toe because of friction.

Expected rocking strength of walls or piers is determined using Equation 7-4, which was derived by taking moments about the toe of the component. The 0.9 factor accounts for a slight reduction in the lever-arm distance to represent the centroid of the vertical compressive stress. If the component is a cantilevered shear wall, the vertical axial compressive force is assumed to act at the center of the wall at the top, which is the reason for an $\alpha$ term equal to 0.5. If the component is a pier, the vertical force is assumed to act near its edge as the pier rotates and the superstructure remains horizontal. The vertical compressive force, $P_{CE}$, should be the best estimate of the gravity force during the earthquake.

Lateral strength of newly constructed masonry walls or piers shall follow the *NEHRP Recommended Provisions* (BSSC, 1995).

**B. Lower Bound Lateral Strength of Walls and Piers**

Lateral strength of walls or piers based on diagonal tension strength is determined using Equation 7-5, which is taken from Turnsek and Sheppard (1980). This equation is only applicable for the range of $L/h$ between 0.67 and 1.00. Because tests do not exist for masonry diagonal tension strength, the bed-joint shear strength, as measured with the in-place shear test, may be substituted where it is assumed that the lower bound diagonal tension strength is equal to the expected value of the bed-joint strength.

Lateral strength limited by toe compression stress is determined using Equation 7-6, which was derived from Abrams (1992). The equation is only applicable for walls or piers loaded with a lateral force that will not result in rocking about their toe. It applies generally to walls with $L/h$ aspect ratios between 1.0 and 1.5 and large vertical compressive stresses. For a lower bound strength, a low estimate of vertical compressive force, $P_{CL}$, must be used. The limiting compressive stress is conservatively taken as 93% of the lower bound masonry compressive strength, $f'_m$. Because the lower bound strength is not determined per Section 7.3.2.1, it may be estimated as a fraction of the expected compressive strength, $f_{me}$.

**C. Lower Bound Vertical Compressive Strength of Walls and Piers**

The lower bound vertical compressive strength given by Equation 7-7 includes a reduction factor equal to 0.85 to relate prism strength to wall strength, and another factor equal to 0.80 for accidental eccentricities.

### C7.4.2.3 Deformation Acceptance Criteria

Unreinforced masonry walls or piers loaded parallel to their plane may experience distress conditions of:

- Minor diagonal-tension or bed-joint cracking
- Major shear cracking or spalling of units
- Loss of strength
- Dislodgment and falling of units
- Out-of-plane movement as a result of excessive rocking

The deformation acceptability criteria given in Section 7.4.2.3 are intended to limit damage accordingly for the goals of each Performance Level.

**A. Linear Procedures**

For the Linear Static Procedure, *m* factors are given for primary and secondary components for each performance level in Table 7-1.

As discussed in Section C7.4.2.1B., nonlinear force-deflection behavior of unreinforced masonry shear walls has been studied experimentally by a number of researchers. Based on many of these wall tests, and subjective but conservative interpretations of the test data, the *m* factors given in Table 7-1 have been derived. Because the experimental research is by no means sufficiently complete to justify directly every combination of wall aspect ratio and vertical compressive stress, the *m* factors have been calibrated in terms of an approximate value for a square wall panel with a nominal amount of vertical compressive stress. Therefore, for the Life Safety Performance Level, an *m* value equal to 3.5 was established as a control point for development of the table. This value is credible considering that the test data revealed ductilities in excess of five for wall panels with similar characteristics.

Variable *m* factors are given for each Performance Level, corresponding to approximate inelastic deflections associated with specific damage states. For Immediate Occupancy, some cracking can be tolerated for typical occupancy conditions; *m* factors range from 1.0 for bed-joint sliding to 1.5 times the height-to-length aspect ratio for a rocking mechanism. Larger nonlinear displacements can be tolerated for rocking piers because bed-joint cracks in rocking components will close after an earthquake, whereas head-joint cracks resulting from bed-joint sliding will not close fully after the sliding stops. The height-to-length aspect ratio is included in the m factor for rocking piers to relate rigid-body rotation of a component to the lateral deflection at the top of the component. The Life Safety Performance Level is related to lateral deflections associated with the dislodgment of masonry units and/or severe cracking; *m* factors are conservatively set at a value of 3.0 for bed-joint sliding or rocking of square wall or pier components. The Collapse Prevention Performance Level is related to a loss of lateral strength for primary components, and unstable gravity-load behavior of secondary components; therefore, *m* factors are approximately one-third larger than for Life Safety.

#### B. Nonlinear Procedures

Nonlinear deformation capacities for primary and secondary components are represented in Figure 7-1 with dimensions *d* and *e* respectively. These values are consistent with the *m* values defined for each Performance Level in Table 7-1, and have been extracted from experimental studies on unreinforced masonry walls as discussed in the previous section. The wall drift before strength is lost (the *d* dimension in Figure 7-1) is equal to 0.4% for bed-joint sliding or rocking of square wall or pier components, which is comparable to laboratory test values of approximately 1% for walls that are governed by these deformation-controlled actions. Drift levels have been reduced substantially to 0.10% for walls with zero vertical compressive stress because rocking or bed-joint sliding mechanisms cannot be mobilized, and, as a result, behavior will be governed by force-controlled actions such as diagonal tension.

## C7.4.3 URM Out-of-Plane Walls

Walls resisting lateral forces normal to their plane are termed “out-of-plane walls.”

### C7.4.3.1 Stiffness

Out-of-plane URM walls not subjected to significant vertical compressive stress, and with no restraint at boundaries for formation of arching mechanisms, do not have a nonlinear range. They are brittle elements that will crack under light lateral forces. Depending on the particular Performance Level, cracking of a wall panel may be acceptable if it can be shown that the wall segments rotating about their ends will be stable under dynamic loading.

The stiffness of walls bending about their weak axis is three or more orders of magnitude less than the stiffness of walls bending about their strong axis. Thus, in an analysis of a building system with walls in each direction, the stiffness of the transverse walls will be much less than that of the in-plane walls and can therefore be neglected.

### C7.4.3.2 Strength Acceptance Criteria

Out-of-plane walls do not need to be analyzed using the Linear Static Procedure because they act as isolated elements spanning across individual stories. Rather than design on the basis of an equivalent base shear applied to the global structural system (per Equation 3-6 with the Linear Static Procedure), out-of-plane walls should resist inertial forces that are prescribed in Section 2.11.7 without cracking for the Immediate Occupancy Performance Level. For similar reasons, the nonlinear procedures are also not applicable for out-of-plane walls.

The expected demand forces depend on response of the floor or roof diaphragms and the in-plane walls. In addition to the transverse inertial forces resulting from the panel weight, a wall panel must also resist deformations resulting from differential lateral drift across a story, as well as diaphragm deflections. These imposed deflections on the out-of-plane wall panels can be accommodated with cracking of the bed-joints if such cracking is determined to be acceptable for the Performance Level. Even under small amounts of vertical compressive stress, cracked panels will remain stable as they deflect with the attached floor or roof diaphragms.

The out-of-plane response of URM walls may be governed by the development of arching mechanisms in the vertical direction between the floor slabs above and below, or in the horizontal direction between columns, pilasters, or walls running in the normal direction. The type of response mechanism for the out-of-plane wall components is sensitive to the conditions at the panel boundaries and the eccentricities of any applied vertical loads. A rigorous analysis requires knowledge of:

- Accelerations of diaphragms above and below the wall panel
- Edge restraint provided by slabs, beams, or spandrels above and below the wall panel, and by columns, pilasters, or walls at each side of the wall panel
- Masonry compressive strength
- Mortar joint tensile strength
- Eccentricity of vertical compressive loads and amounts of vertical load

In spite of these complexities, the out-of-plane strength of URM walls may be bounded as follows.

- The lower limit of strength is defined for a wall panel with no axial load other than its self weight, no edge confinement from stiff elements above, below, or to the sides, no continuity with adjacent wall panels, and low tensile strength. If such conditions are present, the out-of-plane static strength and stiffness may be considered negligible. However, the panel may be stable under dynamic action for the Life Safety and Collapse Prevention Performance Levels, as the weight of the panel tends to restore lateral response back to its original position.
- The upper limit is defined for a wall panel that is ideally fixed in one or two directions by walls, columns, or pilasters that do not deflect, and vertical compressive forces are applied concentrically about the wall panel. Neglecting masonry tensile strength, flexural cracking will commence when a uniform transverse load, $q_{cr}$, is applied equal to:

$$q_{cr} = \frac{2Pt}{h^2} \tag{C7-3}$$

where $P$ is the vertical compressive load, and $h$ and $t$ are the panel height and thickness. Because of arching action, the panel can sustain transverse loads with a reasonable upper bound of:

$$q_{cr} = \frac{6Pt}{h^2} \tag{C7-4}$$

At the maximum load level, the wall stiffness can be considered to be negligible; the structural integrity of the panel is dependent on dynamic stability.

#### C7.4.3.3 Deformation Acceptance Criteria

Acceptance criteria for the Life Safety and Collapse Prevention Performance Levels are based on stable response after cracking of a wall panel has occurred. In addition to the transverse inertial forces resulting from the panel weight, a wall panel must also resist deformations resulting from differential lateral drift across a story, as well as diaphragm deflections. These imposed deflections on the out-of-plane wall panels can be accommodated with cracking of the bed-joints. Even under small amounts of vertical compressive stress, cracked panels will remain stable as they deflect with the attached floor or roof diaphragms. Out-of-plane response of cracked wall panels can be modeled analytically with a dynamic analysis that implicitly considers the motion input at the base of the wall and at the top of the wall. Both the ground motion and the motion of the diaphragm attached to the wall must be determined for this analysis. Research (ABK, 1981) has shown that wall segments should remain stable if their h/t ratio is less than particular values. The values given in Table 7-3, taken from Table C7.4.7.1 of BSSC (1992), are quite conservative relative to the values found in the ABK research. If the h/t ratio of an existing wall exceeds the values given in Table 7-3, and a dynamic stability analysis is not done, then the wall can be either braced (see Section 7.4.1.3I) or thickened with shotcrete (see Section 7.4.1.3C) or a surface coating (see Section 7.4.1.3D). Conversely, the wall may be reinforced (see Section 7.4.1.3E) and analyzed as a reinforced wall, or the wall may be prestressed (see

Section 7.4.1.3F) to increase its cracking moment capacity.

## C7.4.4 Reinforced Masonry In-Plane Walls and Piers

This section applies to reinforced wall and pier components that resist lateral force parallel to their plane. Information on modeling lateral stiffness and expected strength of these components is given for flexural, shear, and axial compressive actions.

As for unreinforced masonry wall and pier components (Section 7.4.2), criteria for solid cantilevered shear walls are expressed in the same context as for individual piers between openings in a perforated shear wall.

### C7.4.4.1 Stiffness

#### A. Linear Elastic Stiffness

Before initial cracking, behavior of reinforced wall or pier components is essentially the same as for unreinforced components, because the reinforcing steel is strained at very low levels and the effective area of masonry in tension is usually quite large relative to that of the reinforcing bars. In this range, lateral stiffness of wall or pier components may be determined assuming a linear elastic analysis of components comprising homogeneous materials. Equations C7-1 and C7-2 may be used to determine lateral stiffness of walls and piers, respectively, based on gross uncracked sections and expected elastic moduli of masonry.

For a wall or pier component with sufficient shear strength, flexural cracking will commence at lateral force levels that are a fraction of the ultimate strength. The fraction will depend on the relative amounts of vertical reinforcement and masonry, the reinforcement yield stress, the masonry compressive strength, the length-to-height aspect ratio of the component, and the amount of vertical compressive force. As a result of flexural cracking, the lateral stiffness will reduce, since the masonry is no longer effective in tension. This reduction in stiffness will, however, result in an essentially linear-elastic behavior, provided that the masonry compressive stress remains at approximately one half or less of the ultimate strength and the reinforcement does not yield. Thus, lateral stiffness may be represented with a reduced value representing the effective cracked section.

#### B. Nonlinear Behavior of Reinforced Masonry Walls and Piers

Reinforced walls are known to soften when cracks initiate. Vertical reinforcement becomes effective after flexural cracks develop along mortar bed joints. With further increase in lateral force, the vertical reinforcement may yield, provided that adequate shear strength is provided. The yielding steel will dissipate substantial seismic energy. In such case, inelastic deflection capacity will be limited by the ultimate compressive strain in the masonry at the wall toe as the steel strains reach well beyond their proportional limit.

Upon unloading, wall stresses will be relieved, but deflections will not reduce substantially because cracks will remain open. When force is reversed in direction, the closing of previously opened cracks will be restrained by the reinforcement acting in compression. In this stage, the resistance of the section is primarily from the reinforcement, and the stiffness will reduce suddenly when the load is reversed. When cracks close fully, the element stiffens, and resumes its character from the loading portion of the previous half cycle. The closing of cracks in the load reversal region causes a “pinching” of the hysteretic loop, which reduces the amount of energy dissipation, and increases the element flexibility. After the first large-amplitude cycle, conventional principles of mechanics used for elements subjected to monotonically increasing loadings cannot be used, because deformations in the masonry and the steel, and at their interface, cannot be estimated reliably. Approximate methods must be used to estimate stiffness and deflection capacity.

Nonlinear behavior of RM wall components has been studied, with large-scale experiments done on: (1) single story walls (Shing et al., 1991), (2) two-story walls (Merryman et al., 1990; Leiva and Klingner, 1991), and (3) a five-story building (Seible et al., 1994). Dynamic testing of reduced-scale, reinforced concrete masonry shear wall buildings by Paulson and Abrams (1990) revealed substantial ductility and inelastic energy dissipation.

#### C. Lateral Stiffness with Linear Procedures

The stiffness of RM wall and pier components that are cracked can be an order of magnitude less than those components that are uncracked. Because the length of masonry walls in typical buildings can vary, some walls are likely to crack while others remain uncracked. Therefore, lateral stiffnesses should be based on the consideration of whether individual components will crack or not when subjected to expected amounts of vertical and lateral force. This distinction is important when: (1) distributing story shear force to individual walls, or shear force to adjacent piers in a perforated shear wall, (2) estimating nonlinear force-deflection relations for wall or pier components with the Nonlinear Static or Dynamic Procedures, or (3) determining spectral accelerations based on periods of vibration for the Linear Dynamic Procedure.

The following criteria may be used to determine the uncracked or cracked condition states as stated in Section 7.4.4.1.

$$\text{if } Q_{UF} < M_{cr} \text{ then } I = I_g \tag{C7-5}$$

$$\text{if } Q_{UF} \geq M_{cr} \text{ then } I = I_e \tag{C7-6}$$

where:

$$M_{cr} = f_{te} S_g \tag{C7-7}$$

and:

$f_{te}$ = Expected masonry tensile strength per Section 7.3.2.3

$I_e$ = Effective moment of inertia based on cracking

$I_g$ = Moment of inertia based on the uncracked net mortared/grouted section

$Q_{UF}$ = Estimate of the maximum lateral force that can be delivered to the component as defined with Equation 3-15

$S_g$ = Section modulus for the uncracked net mortared/grouted section

The stiffness of a cracked reinforced component can be determined based on a moment-curvature analysis of a particular wall or pier cross section, recognizing the amount and placement of vertical reinforcement, the relative elastic moduli for the masonry and reinforcement, and the expected amounts of axial force and bending moment. Alternatively, the secant stiffness of a cracked reinforced component can be determined using Equation C7-8.

$$\frac{I_e}{I_g} = \left[\frac{15,000}{f_{ye}} + \frac{f_a}{f_{me}}\right]\left[\frac{1}{1 + 0.75(L/h_{eff})^2}\right] \tag{C7-8}$$

where:

$f_a$ = Expected amount of vertical compressive stress based on load combinations given in Equations 3-1 and 3-2

$f_{me}$ = Expected masonry compressive strength as determined per Section 7.3.2.1

$f_{ye}$ = Expected reinforcement yield stress as determined per Section 7.3.2.6

$h_{eff}$ = Height to resultant of lateral force

$L$ = Wall or pier length

Using Equation C7-8, the effective moment of inertia can be determined without considering the amount of lateral force or extent of cracking. This simplification avoids any iterations related to the interaction of demand forces and stiffnesses—a cumbersome process, particularly for deformation-controlled elements where the elastic demand forces, $Q_E$, are fictitious, as discussed in Section C7.4.2.1C. The derivation for Equation C7-8 can be found in Priestley and Hart (1989).

### C7.4.4.2 Strength Acceptance Criteria for Reinforced Masonry

The requirements of Sections 7.4.4.2A, 7.4.4.2B, and 7.4.4.2C are based on the latest revisions to the *NEHRP Recommended Seismic Provisions for New Buildings* (BSSC, 1995) for design of newly constructed reinforced masonry shear walls. The same assumptions, procedures, and requirements are intended for existing wall or pier components.

The lateral strength of RM wall or pier components is governed by either flexural or shear action. The ultimate limit state for flexural action is masonry compressive strain at the wall toe, or tensile fracture of vertical reinforcement. Shear strength is limited by yielding of horizontal shear reinforcement, which causes diagonal tension cracks to widen and, in so doing, reduces aggregate interlock mechanisms. A flexural mechanism should be considered as a deformation-controlled action because it involves yielding of reinforcement and some significant levels of inelastic deformation capacity. Assumptions and procedures for determining expected lateral strength of RM shear walls are given in Section 7.4.4.2A for flexure.

A shear mechanism should be considered as a force-controlled action because it involves diagonal tension of masonry. Assumptions and procedures for determining the lower bound lateral strength of RM shear walls are given in Section 7.4.4.2B.

The resistance of RM walls to vertical compressive stress should be considered as a force-controlled action, and should be characterized by the lower bound strength given in Section 7.4.4.2D.

#### A. Expected Flexural Strength of Walls and Piers

Expected flexural strength of wall or pier components shall be based on assumptions given in this section, which are similar to those used for strength design of reinforced concrete.

#### B. Lower Bound Shear Strength of Walls and Piers

Lower bound shear strength of RM wall or pier components is limited to values given by Equations 7-9 and 7-10 for different moment-to-shear ratios. The expected value of masonry compressive strength shall be used to determine these limiting shear forces, which are also considered to be expected values.

Shear resistance is assumed attributable to the strength of both the masonry and reinforcement.

The previous criteria in the *NEHRP Recommended Provisions* (BSSC, 1995) for shear in a plastic hinge zone have been waived, since Equation 7-11 for masonry shear strength is based on tests of shear walls (Shing et al., 1991) where the shear was transferred across a plastic hinge zone. Expected masonry compressive strength, $f_{me}$, and expected axial compressive force, $P_{CE}$, are to be used to determine the expected masonry shear strength.

The lower bound shear strength attributable to the horizontal reinforcement is given by Equation 7-12. The previous form of this equation in the *NEHRP Provisions* (BSSC, 1995) has been revised for clarity to the more familiar format used for concrete members. The limit that $d_v$ not exceed the wall height is intended for squat walls (where $d_v$ is larger than $h$), so that the assumed number of horizontal bars crossing a 45-degree diagonal crack will not exceed the actual number of bars. The 0.5 factor on reinforcement shear strength is taken from research on reinforced masonry shear walls (Shing et al., 1991) and accounts for nonuniform straining of horizontal reinforcement along the component height.

#### C. Strength Considerations for Flanged Walls

Flanges on masonry shear walls will increase the lateral strength and stiffness appreciably; however, they can only be considered effective when the conditions of this section are met.

The width of flange that may be considered effective in compression or tension is based on research done on reinforced masonry flanged walls (He and Priestley, 1992).

#### D. Lower Bound Vertical Compressive Strength of Walls and Piers

Equation 7-13 for lower bound axial compressive strength is similar to that for reinforced concrete columns. Lower bound strengths of masonry and reinforcement shall be used, rather than expected strengths. The 0.8 factor represents a minimum eccentricity of the vertical compressive load.

### C7.4.4.3 Deformation Acceptance Criteria

#### A. Linear Procedures

For the Linear Static Procedure, *m* factors are given for primary and secondary components for each Performance Level in Table 7-4. Factors are given to represent variable amounts of inelastic deformation capacity for (1) various ratios of vertical compressive stress to expected masonry compressive strength, (2) wall or pier aspect ratios, and (3) index values representing amounts of reinforcement, expected yield stress of reinforcement, and expected masonry compressive strength.

The *m* factors were determined from an analysis of lateral deflections for reinforced wall or pier elements based on the three parameters included in the table. Curvature ductilities, $\mu_\phi$, were determined by dividing the ultimate curvature, $\phi_u$, by the curvature at first yield, $\phi_y$, per Equation C7-9.

$$\mu_\phi = \frac{\phi_u M_y}{\phi_y M_u} \qquad \text{(C7-9)}$$

Displacement ductilities, $\mu_\Delta$, were then determined from curvature ductilities, considering plastic rotations at the base of component being limited to a plastic-hinge zone length, $l_p$, equal to:

$$l_p = 0.2L + 0.04h_{eff} \tag{C7-10}$$

which then gave:

$$\mu_\Delta = 1 + 3(\mu_\phi - 1)\left(\frac{l_p}{L}\right)\left(1 - 0.5\frac{l_p}{L}\right) \tag{C7-11}$$

Analytical procedures were based on those presented in Paulay and Priestley (1992).

For the Collapse Prevention Performance Level, *m* factors were assigned equal to these displacement ductilities.

Variable *m* factors are given for each Performance Level, corresponding to approximate inelastic deflections associated with specific damage states. For Immediate Occupancy, some cracking can be tolerated for typical occupancy conditions; *m* factors range from 1.0 to 4.0, depending on the amount of vertical compressive stress, the aspect ratio, and the amount of reinforcement. The Life Safety Performance Level is related to lateral deflections associated with the dislodgment of masonry units and/or severe cracking; *m* factors are approximately twice those for Immediate Occupancy. The Collapse Prevention Performance Level is related to a loss of lateral strength for primary components, and unstable gravity-load behavior of secondary components; *m* factors are approximately one-third larger than for Life Safety.

#### B. Nonlinear Procedures

Nonlinear deformation capacities for primary and secondary components are represented in Figure 7-1 with dimensions *d* and *e*, respectively. These values are consistent with the *m* values defined for each Performance Level in Table 7-4.

Some cracking can be tolerated for Immediate Occupancy. Because of the presence of reinforcement, propagation of cracks will be limited, and thus acceptable wall or pier drifts are larger than those for URM walls.

The Life Safety Performance Level corresponds to severe cracking of the masonry, or a potential for masonry units to dislodge. If spacings of vertical and horizontal reinforcement are equal to or less than 16 inches, these effects will be minimized, and the acceptable drifts contained in Table 7-5 may be increased by 25%.

Severe loss of lateral strength of a wall or pier element can precipitate collapse of a lateral-load or gravity-load structural system. In laboratory experiments, severe loss of strength for in-plane reinforced masonry walls has been observed to occur at lateral drifts exceeding 1.0% for moderate amounts of reinforcement and vertical compressive stress.

## C7.4.5 RM Out-of-Plane Walls

Walls resisting lateral forces normal to their plane are termed “out-of-plane walls.” The stiffness of walls bending about their weak axis is three or more orders of magnitude less than the stiffness of walls bending about their strong axis. If a building system contains walls in both directions, the stiffness of the transverse walls will be insignificant. Analysis of out-of-plane walls with the LSP is not warranted, because out-of-plane walls will not attract appreciable lateral forces. Rather than design on the basis of a pseudo lateral load applied to the global structural system (as in Equation 3-6 with the LSP), out-of-plane walls should resist inertial forces that are prescribed in Section 2.11.7. For similar reasons, the NSP is also not applicable for out-of-plane walls. However, the Nonlinear Dynamic Procedure may be useful for out-of-plane walls not complying with strength criteria based on an equivalent static uniform loading.

### C7.4.5.1 Stiffness

The static behavior and dynamic response of RM walls bending out-of-plane have revealed very large flexibilities and inelastic deformation capacities. Testing of wall panels is reported by Agbabian et al. (1989), Hamid et al. (1989), and Blondet and Mayes (1991). The effect of flexural cracking on stiffness is quite significant, particularly for small percentages of vertical reinforcement. The stiffness of a cracked section can be as low as one-tenth that of the uncracked section.

#### C7.4.5.2 Strength Acceptance Criteria

The strength of reinforced out-of-plane walls is nearly always limited by flexural strength, because the span-to-depth ratio is large.

Reinforced masonry walls usually have a single layer of vertical reinforcement that is centered about a single wythe for hollow-unit masonry, or between two wythes of solid masonry. Nominal ultimate flexural capacity can be calculated assuming a rectangular stress block for the masonry in compression, which results in Equation C7-12 for a section with a single layer of tensile reinforcement.

$$Q_{CE} = M_{CE} = A_s f_{ye} d\left(1 - 0.59\rho\frac{f_{ye}}{f_{me}}\right) \qquad \text{(C7-12)}$$

Tests of RM walls have demonstrated the large inelastic deformation capacity of wall panels subjected to out-of-plane loadings. Deformation capacity is dependent on the amount of vertical reinforcement, the level of vertical compressive stress, and the height-to-thickness aspect ratio.

#### C7.4.5.3 Deformation Acceptance Criteria

Out-of-plane RM walls can resist transverse inertial loadings past the yield limit state with substantial inelastic deformation capacity. If sufficient flexural strength is available to resist the uniform face loading prescribed in Section 2.11.7, and walls are tied to diaphragms at their top and bottom, then they should perform adequately for any level from Immediate Occupancy to Collapse Prevention. Thus, no performance limits are given on out-of-plane deflection of wall panels since post-yield behavior will not need to be relied on.

If the NDP is used, out-of-plane response of the transverse walls may be determined for wall panels performing in the nonlinear range of response. Whereas the out-of-plane walls do not necessarily have to be modeled as part of the global system if strength requirements are met per Section C7.4.3.2, there is no restriction excluding them from a model. On the contrary, inclusion of the out-of-plane walls in a NDP model may be necessary to demonstrate performance for overly slender or weak walls. In such cases, Performance Levels need to be defined in accordance with the estimated out-of-plane deflection of the transverse walls. Because out-of-plane masonry walls are local elements spanning across individual stories or bays, the limit states in the following paragraphs are expressed in terms of lateral deflection across their story height or length between columns or pilasters.

Flexural cracking of an RM wall subjected to out-of-plane bending should occur at the same drift level as for an unreinforced wall. However, this will not, in general, be associated with any Performance Level because cracking of reinforced components is acceptable. As the reinforcement yields at a story drift ratio of approximately 2.0%, cracks will widen substantially and may limit the immediate use of a building.

Life Safety is related to a wall panel reaching its peak strength. This limit state has been estimated to occur at a story drift ratio of 3%, based on experimental research.

The loss of an entire out-of-plane wall may not influence the integrity of the global structural system in the direction under consideration. Therefore, the Collapse Prevention Performance Level should not be applicable for out-of-plane walls. However, the loss of an out-of-plane wall will affect performance of the system in the orthogonal direction when it acts as an in-plane wall. Furthermore, loss of a wall panel can seriously diminish the integrity of the gravity load system if the wall is a bearing wall. Reinforced masonry walls bending out-of-plane are very ductile. Collapse should not occur unless lateral story drift ratios are very large at 5% of the span or larger.

## C7.5 Engineering Properties of Masonry Infills

Masonry infill panels are found in most existing steel or concrete frame building systems. Although they are a result of architectural function, infill panels do resist lateral forces with substantial structural action, and should, therefore, be assumed to be part of the primary lateral-force-resisting system.

Since infill panels are usually placed after floors are constructed, they do not resist gravity dead loads at the time of construction. However, if an infill is in tight contact with the beam above, the panel may help support live loads as well as dead loads from upper stories if they are placed after installation of lower-level infills. In addition, if the masonry infill materials tend to expand with time (as is the case with some clay-unit masonry), and/or the frame columns tend to shrink or creep (the case with concrete columns), an infill panel can attract vertical compressive stress as a portion of the gravity loads are redistributed to it from the frame.

In Section 7.5, infill panels are not considered as secondary elements even if they may support gravity loads, because loss of an infill panel should not jeopardize the vertical-load-carrying system. Typically, frames are designed to resist 100% of gravity forces, and should not suffer a loss in structural integrity if the infill panels are eliminated.

If an infill panel is destroyed during seismic shaking, and falls out from the surrounding frame, collapse of the structural system can still be prevented, assuming that the frame resists the full lateral load. If a lateral-force analysis of the bare frame system demonstrates prevention of collapse, then the infill panels should not be subject to limits set forth by the Collapse Prevention Performance Level.

## C7.5.1 Types of Masonry Infills

The engineering properties given in Section 7.5 are applicable to building systems with existing, enhanced, or new masonry infills that combine to rehabilitate a building system. In addition, the *Guidelines* provide specific recommendations on minimum requirements for enhancement of existing infill panels, in order that their structural properties may be considered the same as new or existing elements.

Stiffness assumptions, strength criteria, and acceptable deflections for various limit states as described in Sections 7.5.2 through 7.5.3 are common for existing or enhanced masonry infills, or new masonry infills added to an existing building system. Principles of mechanics are the same regardless of the age of a masonry element. Physically, there should be no difference in stiffness assumptions, strength criteria, or inelastic behavior for existing, enhanced, or newly constructed infills. Thus, guidelines on determining engineering properties for each of the three fundamental infill types are expressed in common in these sections.

In Sections 7.5.2 through 7.5.3, infill panels subjected to in-plane lateral forces are separated from walls subjected to out-of-plane forces, because their stiffnesses, strengths, and acceptable deformations are quite different. Unreinforced masonry infills are considered since they are the most common. However, RM infills can be considered with the same criteria, since the in-plane and out-of-plane mechanisms are not influenced negatively by reinforcement.

### C7.5.1.1 Existing Masonry Infills

Existing masonry infills will have a significant influence on the lateral strength and drift of a building system. Certain masonry infills may have a brittle character; their removal may improve the overall energy dissipation capabilities of a system, and thus be an acceptable rehabilitation option. When considering a particular rehabilitation scheme, existing masonry infills, or their extraction, should be included in the structural analysis along with any new masonry infill panels that may be added.

A thorough condition assessment should be made of existing masonry infills to increase the level of confidence in characterizing structural properties.

Infilled frame buildings are mostly mid- to high-rise buildings with steel or concrete gravity-load-resisting systems and masonry infill perimeter walls. Steel frame elements are often encased in concrete, brick, or tile for fire protection purposes. For fire protection, masonry infills may also be found within the interior of buildings. Interior infills may extend up to the bottom of beams or slabs, or they may stop at the ceiling level. Floor framing systems in infilled buildings may consist of almost any material. Because infilled frames tend to be significantly stiffer than noninfilled frames, they are likely to be the main lateral-force-resisting elements of the building.

Typical masonry units used for infill panels are clay bricks, concrete blocks, or hollow clay tile. For buildings constructed earlier in this century, masonry units were typically red clay bricks laid in lime mortar. In more recent times, other types of units may have been used, and mortars may have included portland or masonry cement.

Clay-unit infills are common in two or three wythes, and are bonded with headers every five to seven courses. In many cases, the exterior wythe consists of a facing of bricks, decorative terra cotta units, or cast stone (or some combination of these) placed outside the plane of the frame for architectural and weathering purposes. In these cases, the brick wythe is attached to the infill backing with intermittent header bricks or corrugated metal ties placed in the mortar joints. Terra cotta and stone veneers are typically anchored to the infill backing with round metal tie rods bent in the form of staples or hooks.

Location of the infill varies relative to the frame and the connections between infills and frames. Commonly, the interior wythes are supported on top of the beams and the veneer masonry wythe is supported on a steel ledger plate or angle cantilevering out from the beams. In other cases, the outer wythe is supported by the keying action of header bricks interlocked with the interior wythes. The masonry units may be tightly fitted with the surrounding frame units, or gaps may exist between the frame and the infill.

Masonry infills may entirely fill one or more bays and stories in a frame, although this condition is likely only in walls away from the street. More commonly, masonry infills are partial-height infills, or full-height infills with window openings.

Infilled reinforced concrete or steel frames were typically designed to carry all gravity loads and the infills were not intended to be load bearing. Frames were usually not designed for any significant lateral loads. In reinforced concrete frames, beam reinforcement is likely to not be continuous through the joints, and the column bar splices may not be adequate for tension forces. Frame elements may have some widely spaced ties that are not likely to provide adequate shear capacity or ductility.

Steel frames are commonly constructed with rolled shapes for the lighter framing and riveted built-up sections for the heavier framing. Beam connections are usually semi-rigid, with beam seats and clip angles connecting the beam flanges to the column. In some cases, connections with gusset plates may have been used in exterior frames to resist wind loads.

Infilled frames combine nonductile frame systems with brittle masonry materials; hence they conceptually form a poor lateral-load-resisting system. However, observations from the 1906 San Francisco earthquake and other subsequent earthquakes indicate a surprisingly good performance for steel infilled frame buildings. This good performance is attributed to (1) the interaction of the infill with the steel frame, in which the infill provides a significant bracing mechanism for the frame, and (2) the fact that the steel frame members possess adequate ductility to accommodate the demands imposed on them by the infill. In addition, cracking of the infill and the friction between the infill and the frame provides a significant energy dissipation mechanism.

Reinforced concrete infilled frames have not fared as well as steel infilled frames in severe earthquakes, primarily due to the inability of nonductile concrete members to accommodate the demands imposed on them through the interaction with the infill.

Structural frame and masonry infill respond to lateral shaking as a system, both frame and infill participating in the response through a complex interaction. The overall system response and the interaction between frame and infill are influenced by the material and geometric characteristics of each of these elements and the variation of the element characteristics during earthquake response.

The arrangement of infill panels along the height of the building and in plan may have significant influence on the overall earthquake response of the building. This occurs, for example, when framing is kept open at the street side of a building but is infilled along other exterior frames. In this situation, there is the possibility that the resulting asymmetry will produce increased damage due to torsional response of the building. Another case is the lack of infills at a lower story level, which can result in an undesirable soft-story configuration. Similar eccentric or soft-story conditions may be created during the earthquake if infills in a lower story and/or along a side of the building fail, while infill panels in other locations remain relatively undamaged. These overall system concerns can be identified and considered in design if the response behavior of the frame-infill system can be understood and analyzed at the local, single infill panel level.

The failure modes of interest for earthquake performance are as follows.

#### A. Dislodgment of Masonry Units During an Earthquake

This may result from excessive deformations of the infills due to in-plane or out-of-plane forces, or from inadequate anchorage of veneer courses to the backing courses. Where an exterior wythe of masonry extends beyond the structural frame, delamination or splitting at the collar joint may occur under the action of in-plane loads. Because partial infills and infills with openings are more flexible than solid infills, they may be more prone to this type of damage.

#### B. Falling of Infill Panels

Infill panels (or large portions of wall) may fall out of the surrounding frame due to inadequate out-of-plane restraint at the frame-infill interface, or due to out-of-plane flexural or shear failure of the infill panel. In undamaged infills, these failures may result from out-of-plane inertial forces, especially for infills at higher story levels and with a large h/t ratio. However, it is more likely for out-of-plane failure to occur after the masonry units become dislodged due to damage from in-plane loading.

#### C. In-Plane Failure of Infill Panels

Infill panels may lose their strength and stiffness due to in-plane forces imparted to them during earthquake response. This failure mode does not necessarily lead to failure of the overall structural system, although the changes in the strength and stiffnesses of the infill panels are likely to have significant impact on the overall structural response. Also, dislodgment of masonry units or falling of infill panels are likely to follow the failure of the infills due to in-plane deformations. Shear strength of the infilled frame under these circumstances would be expected to be controlled by the shear capacity of the infill. Either of two modes of failure may occur: sliding shear failure along a bed-joint line (commonly about mid-height), or failure in compression of the diagonal strut that forms within the panel.

#### D. Premature Failure of Frame Elements or Connections

The interaction of the frame with the infill during earthquake shaking results in transfer of interactive forces between frame members and the infill at contact areas. These contact forces may generate internal forces in frame members that are significantly different than those determined by considering lateral response of the frame alone (which has been the usual design assumption in the past). Hence, premature failures may occur in the beams, columns, or connections of the frame. Examples of this behavior are the shear failures induced in columns due to reduced effective flexural length—which may occur when masonry infills form only the spandrels above and below continuous window openings (“captive columns”)—and failures of columns, beams, and connections due to compressive “strut” reactions imparted to them by the masonry infill. Another mode of failure of frame elements is the failure of the tension or compression chords of the infill frame acting as a monolithic flexural element. This mode may predominate in cases where the infill frame is relatively slender and, in particular, where a single bay is infilled in a multibay, multistory building. In this case, the infill frame may act effectively as a flexurally controlled shear wall, with the infill acting as the web and the boundary columns acting as tension and compression chords. Strength in this mode is calculated by conventional flexural procedures, considering the possibility of failure of either the tension chord or the compression chord. Due consideration should be given to tension chord splices, and to tension chord and compression chord offset bars.

#### E. Failure of the Frame

Upon complete failure of the infill system—provided that no premature failure of the frame elements has occurred—the structural response and performance are determined by the characteristics of the frame only (except, perhaps, for the contribution of the damaged infills to structural damping). As noted above, falling infills present a hazard in themselves, and may also produce a fundamental change in the response of the infill structure. The response of the frame with the infill missing should be assessed, keeping in mind the likelihood that a soft story configuration or stiffness eccentricity may have resulted.

### C7.5.1.2 New Masonry Infills

Newly constructed masonry infill panels can be added to an existing building system for the purpose of strengthening, stiffening, or increasing inelastic deformation and energy dissipation capacity.

Design of newly constructed masonry infill panels is not addressed by any existing standards. Procedures for estimating strength and stiffness for new infills shall be in accordance with Sections 7.5.2 and 7.5.3.

### C7.5.1.3 Enhanced Masonry Infills

Rehabilitation methods for masonry walls as described in Section 7.4.1.3 are generally applicable as well for masonry infills. In-plane strength and stiffness of a perforated infill panel can be increased by infilling openings with masonry, by applying shotcrete or surface coatings to the face of an infill panel, by injecting grout into the joints, or by repointing mortar joints. Out-of-plane strength can be enhanced with these methods in addition to providing stiffening elements. Enlarging openings is not feasible for an infill panel because panels elements are not susceptible to rocking motions as are masonry piers or walls.

Reinforced or prestressed cores are not practical because vertical coring of an infill panel is difficult.

In addition, the following two enhancement methods are unique to infill rehabilitation.

#### A. Boundary Restraints for Infill Panels

The stability of isolated infill panels with gaps between them and the surrounding frame may be improved by restraining out-of-plane movements with steel fixtures that are anchored to the adjacent frame members. This method does not fill in the gaps, and therefore does not improve in-plane action.

#### B. Joints Around Infill Panels

Infill panels with gaps around their perimeter do not fully participate in resisting lateral forces. Furthermore, such walls require perimeter restraints for out-of-plane forces. By filling gaps around an infill panel, multiple benefits can be gained, including increased in-plane strength and stiffness, increased out-of-plane strength (through arching action), and elimination of the need for out-of-plane perimeter restraints.

## C7.5.2 In-Plane Masonry Infills

Infill panels resisting lateral forces parallel to their plane are termed "in-plane infills."

Behavior of infilled frame systems subjected to in-plane lateral forces is influenced by mechanical properties of both the frame and infill materials, stress or lateral deformation levels, existence of openings in the infill, and the geometrical proportions of the system. Existence of an initial gap between the frame members and the infill also influences the behavior of the system.

### C7.5.2.1 Stiffness

In-plane lateral stiffness of an infilled frame system is not the same as the sum of the frame and infill stiffnesses, because of the interaction of the infill with the surrounding frame. Experiments have shown that under lateral forces, the frame tends to separate from the infill near windward lower and leeward upper corners of the infill panels, causing compressive contact stresses to develop between the frame and the infill at the other diagonally opposite corners. Recognizing this behavior, the stiffness contribution of the infill is represented with an equivalent compression strut connecting windward upper and leeward lower corners of the infilled frame. In such an analytical model, if the thickness and modulus of elasticity of the strut are assumed to be the same as those of the infill, the problem is reduced to determining the effective width of the compression strut. Solidly infilled frames may be modeled with a single compression strut in this fashion.

For global building analysis purposes, the compression struts representing infill stiffness of solid infill panels may be placed concentrically across the diagonals of the frame, effectively forming a concentrically braced frame system (Figure C7-1). In this configuration, however, the forces imposed on columns (and beams) of the frame by the infill are not represented. To account for these effects, compression struts may be placed eccentrically within the frames as shown in Figure C7-2. If the analytical models incorporate eccentrically located compression struts, the results should yield infill effects on columns directly.

Alternatively, global analyses may be performed using concentric braced frame models, and the infill effects on columns (or beams) may be evaluated at a local level by applying the strut loads onto the columns (or beams).

Diagonally concentric equivalent struts may also be used to incorporate infill panel stiffnesses into analytical models for perforated infill panels (e.g., infills with window openings), provided that the equivalent stiffness of the infill is determined using appropriate analysis methods (e.g., finite element analysis) in a consistent fashion with the global analytical model. Analysis of local effects, however, must consider various possible stress fields that can potentially develop within the infill. A possible representation of these stress fields with multiple compression struts, as shown in Figure C7-3, have been proposed by Hamburger (1993). Theoretical work and experimental data for determining multiple strut placement and strut properties, however, are not sufficient to establish reliable guidelines; the use of this approach requires exercise of judgment on a case-by-case basis.

The equivalent strut concept was first proposed by Polyakov (1960). Since then, Holmes (1961, 1963), Stafford Smith (1962, 1966, 1968) Stafford Smith and Carter (1969), Mainstone (1971 and 1974), Mainstone and Weeks (1971), and others have proposed methods and relationships to determine equivalent strut properties. Klingner & Bertero (1976) have found the method developed by Mainstone to provide reasonable approximation to observed behavior of infill panels.

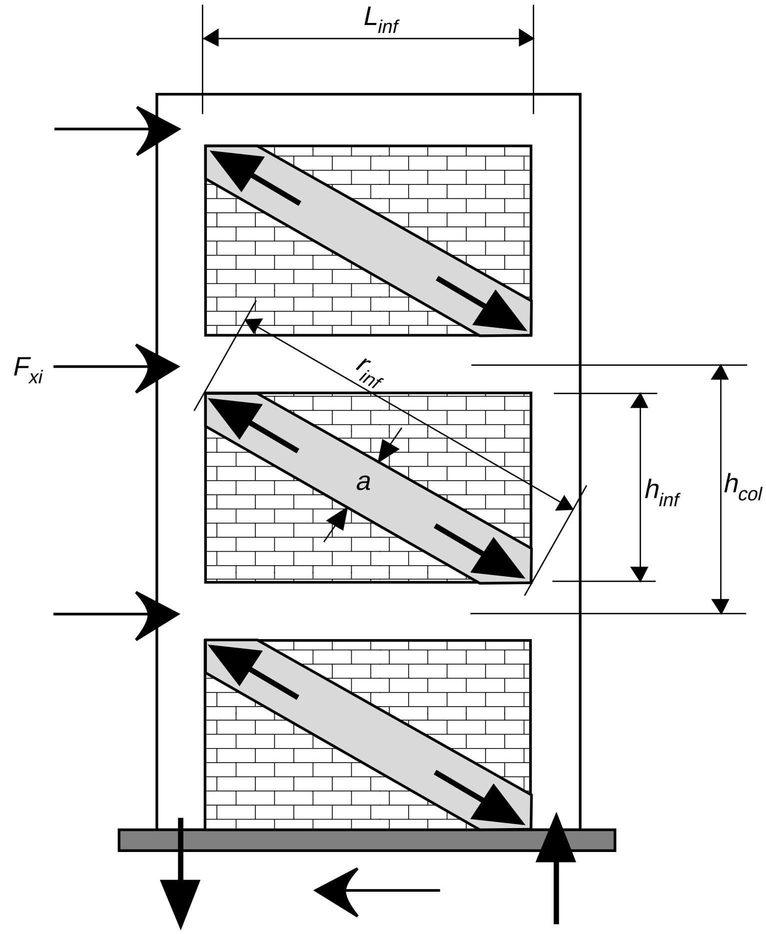


***Figure C7-1 Compression Strut Analogy–Concentric Struts***

Angel et al. (1994) have found a strut width equal to one-eighth of the diagonal dimension of the infill panel to provide good correlation with experimental results; they also proposed modifications to the frame-infill system stiffness expression developed by Holmes to account for the effects of cyclic loading.

In addition to these empirical studies, frame infill systems have been studied using detailed finite element models (Lotfi and Shing, 1994; Durrani and Luo, 1994; Mehrabi and Shing, 1994; Gergely et al., 1994; Kariotis et al., 1994). Although it is not presently practical to use general-purpose finite element software to perform detailed nonlinear finite element analyses of infill frames, recently developed special-purpose computer software, such as FEM/I (Ewing et al., 1987) may be used to determine equivalent strut properties from nonlinear finite element analyses of typical frame-infill configurations. With such special purpose software, the force-deformation behavior of the frame-infill system is determined through nonlinear finite element analysis, and the equivalent strut properties for use in elastic models are derived from the force-deformation relationship for a target displacement.

Experimental studies done at the Y-12 Plant of the Oak Ridge National Laboratory (Flanagan et al., 1994) showed that the same equivalent strut modeling procedures could be used for infill panels constructed with hollow-clay tile.

In the *Guidelines*, the equivalent compression strut model is adopted to represent the in-plane stiffness of solid masonry infill panels. The relationship used to determine the strut width, Equation 7-14, has been proposed by Mainstone (1971). There are not sufficient data to provide modeling guidelines for representing stiffness of perforated infill wall panels with multiple

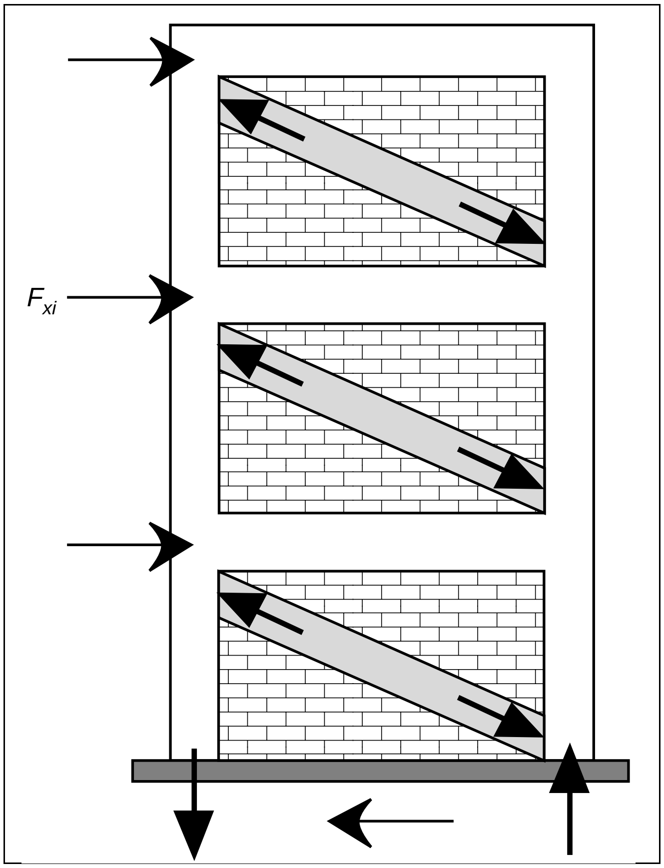


*Figure C7-2* ***Compression Strut Analogy–Eccentric Struts***

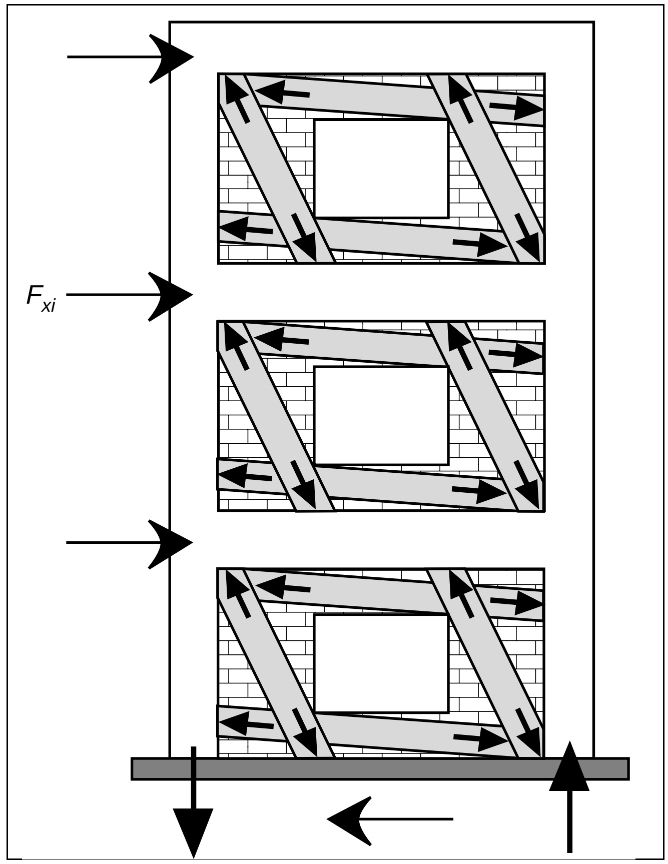


*Figure C7-3* ***Compression Strut Analogy–Perforated Infills***

equivalent struts. However, as discussed above, equivalent struts may still be used in analyses of infilled frames with perforated infills, provided that the equivalent strut properties are derived from detailed finite element analyses of representative frame-infill systems.

### C7.5.2.2 Strength Acceptance Criteria

#### A. Infill Shear Strength

The horizontal component of the force resisted by the equivalent strut should be compared with the expected shear strength of an infill panel times the appropriate *m* and κ factors per the load combination given in Equation 3-18.

The expected infill strength as given with Equation 7-15 is based on an average shear stress across the net mortared/grouted area of a horizontal section cut across the panel. The expected shear strength across this area, $f_{vie}$, is taken as the expected bed-joint shear strength, $v_{me}$, for existing construction, or values based on the 1994 *NEHRP Recommended Provisions* (BSSC, 1995) for new construction. No allowance is made for shear strength enhancements due to vertical compressive stress, because gravity forces are assumed to be resisted by the frame.

The expected infill shear strength is based on bed-joint sliding with no confinement from the surrounding frame, and may thus be less than the actual shear strength. A study done by Angel et. al. (1994) found that results from in-place shear tests provide a conservative estimate of infill shear strength. A resolution based on discussions at an NCEER Workshop on Masonry Infills (Abrams, ed., 1994) was that average infill shear stress provided a good index of lateral infill shear strength.

#### B. Required Strength of Column Members Adjacent to Infill Panels

Infill panels can attract substantial forces to adjacent frame members. These forces can be more demanding of the strength and inelastic deformation capacity of beam and column members than those resulting from lateral design forces applied to a bare frame. Because a stiff masonry infill panel can attract more lateral force than a frame can resist, frames must be checked to see if they are capable of resisting infill forces in the ductile manner that is assumed for their design or evaluation.

Shear strength of the column members should be checked to resist either the horizontal component of the axial force in equivalent struts, or the shear forces resulting from development of plastic hinges at the top and bottom of a column of reduced height. Although neither of these two conditions is exactly representative of what may occur—because of the complex interactions between a frame and an infill panel—these criteria should result in an adequate check to insure ductility of the frame.

The first condition is depicted in Figure C7-4, where the equivalent strut is assumed to be acting eccentrically about the beam-to-column joint with the action illustrated in Figure C7-2. For simplicity, the strut force is assumed to be applied to the column member at the edge of its equivalent width, *a*. This assumption results in a short shear span of the column equal to $l_{ceff}$, for which the horizontal strut component must be resisted over. The infill force applied to the frame should be an expected value and not an unreduced elastic demand force as determined with the LSP. The strength of the column member is also an expected strength. Thus, the relative *m* factors for both the column and the infill panel should be considered when checking the column strength for this action.

Because the first condition can result in excessively high column shear forces, a second option is based on achieving ductile performance of the column when partially braced by the infill panel. This second option consists of checking column shear strength for resisting expected flexural strengths applied at the top and bottom of a short column portion of height $l_{ceff}$. This requirement may lead to smaller shear forces for relatively light column flexural strengths and will insure that hinging of the column members will occur. The same condition shall be applied to captive columns braced with partial height infills.

Effects of infill panels on frames may be neglected if the bed-joint shear strength of masonry is known to be sufficiently low. In this case, the infill panel will conform to the deflected shape of the frame as courses of masonry slide relative to one another across bed joints. The limit of 50 psi for expected masonry strength defines this sufficiently weak condition, which must be determined from in-place shear tests.

#### C. Required Strength of Beam Members Adjacent to Infill Panels

For the same reasons as discussed for column members adjacent to infill panels in the preceding section, flexural and shear strengths of beam members must be checked to ensure the transfer of eccentric infill vertical force components. Again, two options are given to check either strength or deformation capacity of the beam. Equations 7-18 and 7-19 are based on the geometry of forces as shown in Figure C7-5.

### C7.5.2.3 Deformation Acceptance Criteria

#### A. Linear Procedures

In Table 7-6, *m* factors are given only for infill panels acting as primary elements. Because the surrounding frame is assumed to resist gravity forces, the only structural role of the infill is to resist lateral forces, which is a primary action. Thus, infill panels are not considered to act as secondary members and do not need to be checked for their ability to support gravity loads while deflecting laterally.

No *m* factors are given in Table 7-6 for the Collapse Prevention Performance Level because loss of an entire infill panel should not result in collapse of the frame system. In this case, component behavior is not related to performance of the system. However, the ability of the bare frame to resist gravity and lateral forces must be checked to see if collapse will be prevented.

Amounts of inelastic deformation for an infill panel are expressed in terms of a $\beta$ factor that expresses the relative frame to infill strength. When the expected lateral frame strength exceeds approximately 1.3 times the expected shear strength of an infill, any sudden loss of infill strength is not likely to result in a substantial decrease in lateral strength of the frame-infill system. Furthermore, when the frame is strong relative to the infill, it will offer more confinement to the infill because inelastic deformations of frame members will be minimized. When the expected strength of the frame is approximately less than 0.7 times the infill expected

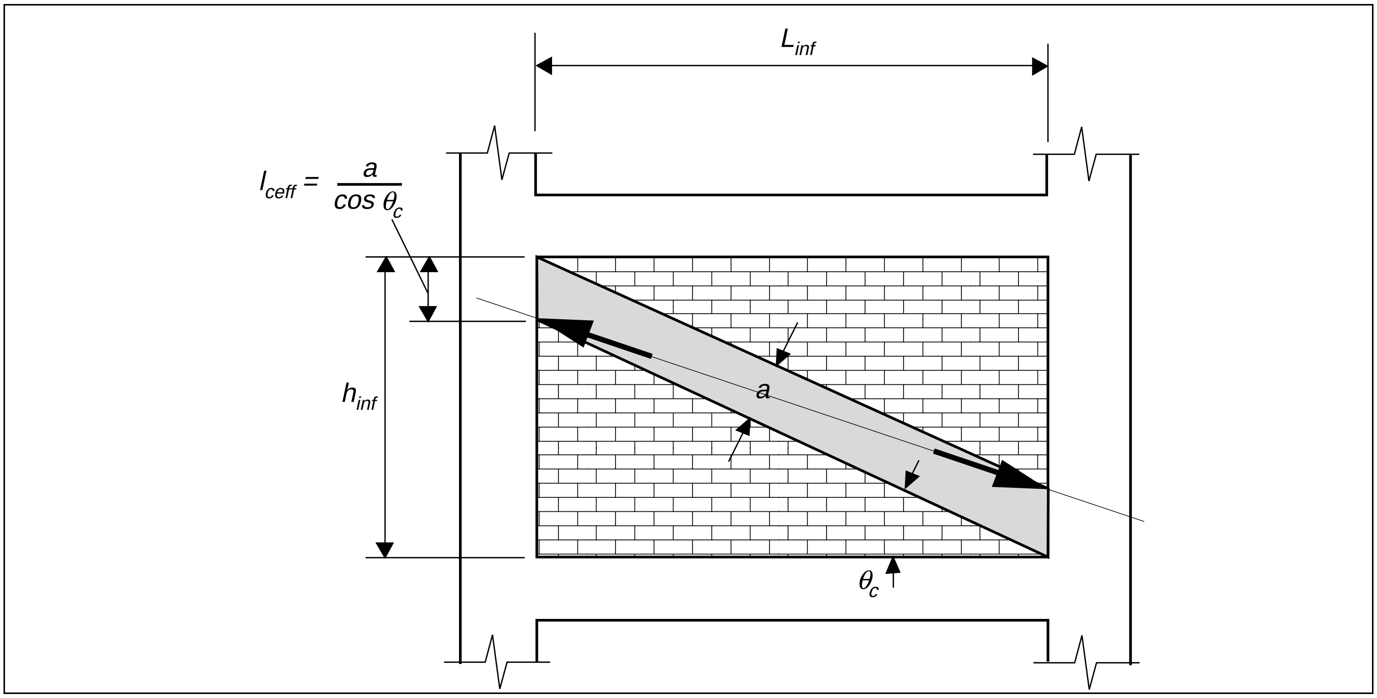


*Figure C7-4 Estimating Forces Applied to Columns*

strength, a sudden loss of infill strength may result in a sudden and substantial decrease in strength of the frame-infill system. Also, when the frame is weak relative to the infill, confinement effects will be reduced as inelastic deformations of frame members occur.

Inelastic deformation capacity of infills is also expressed in terms of the length-to-height aspect ratio of an infill panel. Larger *m* factors are given for more slender panels than stocky panels because they will be more flexible and thus more adaptable to frame distortions. For taller panels, the angle of the equivalent strut relative to the horizontal will be larger than for stocky panels, and thus offer less resistance to lateral forces.

For the Immediate Occupancy Performance Level, some minor cracking of an infill panel is permissible, and thus *m* factors in Table 7-6 are larger than one inferring that some inelastic deformations can occur. However, when the frame strength is low relative to that of the infill, cracking of the infill can result in damage to the adjacent frame, which could alter the performance of the frame-infill system. Thus, for low $\beta$ values, the *m* values should be limited to 1.0. For systems with moderate or large $\beta$ values, no distinction is made in *m* values for the relative frame-to-infill strength because this level of infill should not result in damage to frame members.

### B. Nonlinear Procedures

In Table 7-7, inelastic deformation capacities of masonry infill panels are expressed with the *d* dimension, which is given in terms of the generalized force-deflection relations as depicted in Figure 7-1. No values for terms *c* or *e* are given in the table because they apply only to secondary elements. For the reasons discussed in the previous section, infill panels are considered only as primary elements.

Deformation capacity and acceptable deformations are expressed in terms of the relative frame-to-infill strength and the panel aspect ratio, as is done with the *m* factors in Table 7-6.

At a very low level of story drift ratio (on the order of 0.01%), the leeward column of an infilled frame will separate from the infill, resulting in a sudden loss of stiffness. This limit state is of little concern, since the gap will not be visible following the earthquake, and the analysis should have neglected any tension across the gap by using a compression strut. For such a case, the initial stiffness should be based on the axial stiffness of the equivalent strut with properties as defined with

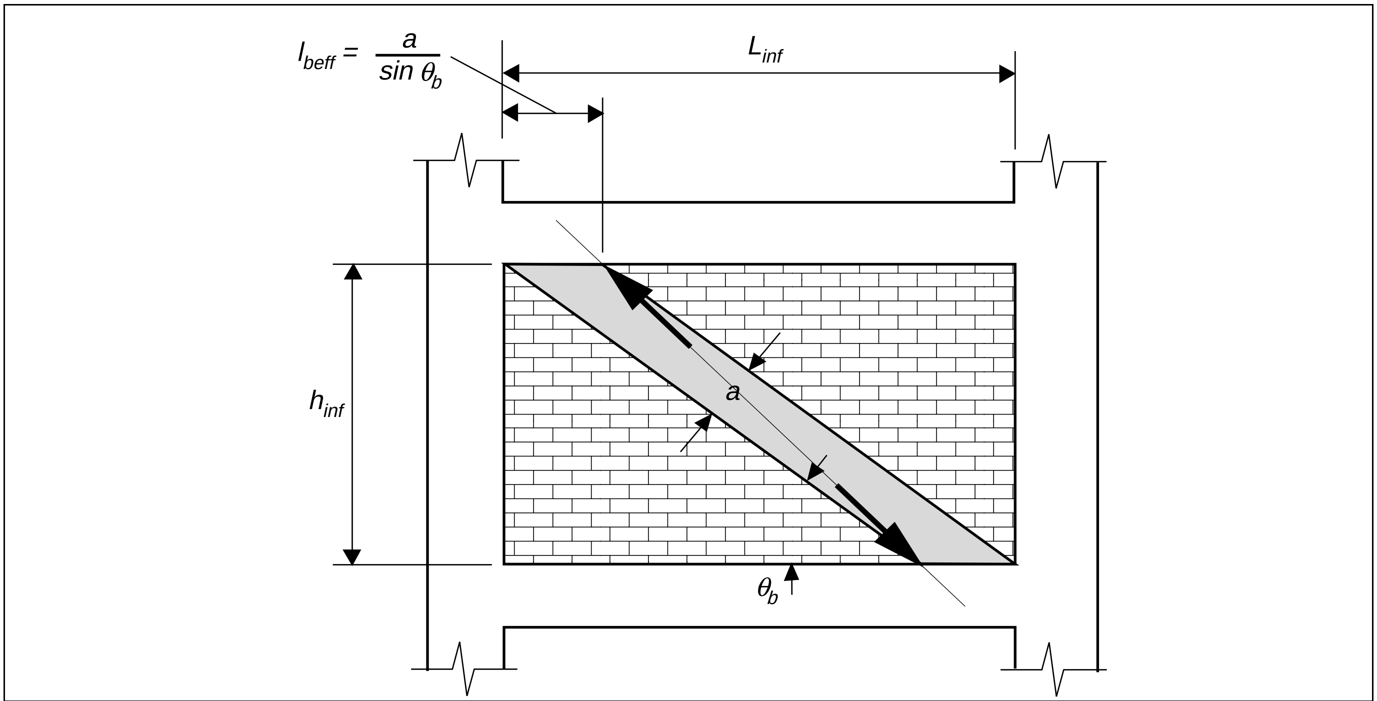


***Figure C7-5 Estimating Forces Applied to Beams***

Equation 7-14, rather than on a fully uncracked solid panel with full contact with the frame on all edges.

As the infill shear stress is increased, minor cracking along bed joints will develop for weaker mortars, or diagonal cracks will form across a panel for stronger mortars. This will occur at a story drift ratio of nominally 0.1% for square panels. Initial cracking of an infill panel will result in a decreased stiffness, but the panel will still continue to resist increased shear forces if confined by the surrounding frame. Following an earthquake, these minor cracks may be noticeable, but no structural repair would be necessary.

Further loading will result in a wider dispersion of bed joint cracks, or an elongation of diagonal cracks. Moderate or severe cracking of a square masonry infill panel can be expected at story drift ratio levels of approximately 0.3% or more. Even in this condition, an infill panel may continue to provide resistance if the surrounding frame is in tight contact and can provide confinement to the masonry assemblage.

Life Safety corresponds to reaching the peak infill strength. In some cases, Life Safety may also be related to dislodgment and falling of masonry units because of the hazard to life or the blocking of egress. For this case, the relative frame-to-infill strength becomes a significant parameter because post-cracked behavior of a masonry infill panel is very much dependent on the confinement offered by the frame.

Experimental studies done at the Y-12 Plant of the Oak Ridge National Laboratory (Flanagan et al., 1993) showed that the same force-deflection properties could be used for infill panels constructed with hollow-clay tile.

## C7.5.3 Out-of-Plane Masonry Infills

Infill panels resisting lateral forces normal to their plane are termed “out-of-plane infills.” The minimum height-to-thickness ratios given in Table 7-8 are based on achieving a transverse infill strength based on an arching action model that will exceed any plausible acceleration level for each of the various seismic zones.

### C7.5.3.1 Stiffness

The stiffness of infill panels bending about their weak axes is three or more orders of magnitude less than the stiffness of panels bending about their strong axes. Thus, in an analysis of a building system with infills or walls in each direction, the stiffness of the transverse infills can be neglected.

The out-of-plane deflection of an infill panel can be approximated by considering strips of unit width spanning either vertically between floors or horizontally between columns. The uncracked stiffness of the strip can be considered if the maximum bending moment is less than the cracking moment. Post-cracked behavior can be tolerated, provided that conditions exist for arching action to take place.

The restrictions on when arching action can be considered are based on the ability of the panel to develop internal thrusts when being loaded transversely. The panel must be in tight contact with the surrounding beam and column members. These members must have a flexural stiffness sufficiently high so that they will not flex when subjected to the infill thrust forces, as well as a flexural strength large enough to resist the thrusts.

Slender panels may snap through the frame, particularly if ultimate masonry compressive strains are large at their boundaries. Studies done by Angel et al. (1994) have shown that this may occur for panels with a $h_{inf}/t_{inf}$ ratio exceeding 20 if the ultimate strain is 0.005. This slenderness has been set as a limit on when arching action may be considered.

Transverse deflections at mid-length of a one-way strip for panels that will not snap through the frame can be determined with Equation 7-20, which is a simplified version of an equation given by Abrams et al. (1993) assuming arching action and an ultimate masonry compressive strain equal to 0.004.

### C7.5.3.2 Strength Acceptability Criteria

Out-of-plane infills should not be evaluated using the Linear or Nonlinear Static Procedures of Chapter 3 because these infills act as isolated elements spanning across individual stories. The transverse strength of infill panels should exceed the maximum plausible lateral inertial forces that result from the mass of the panel accelerating. Because the evaluation of out-of-plane infill panels does not depend on an unreduced value of base shear—as is done for in-plane components per the LSP—there is no need to use expected values of strength. Thus, strength criteria given in this section are based on lower bound estimates of strength. Actual transverse strengths can be higher.

Masonry infill panels must be restrained perpendicular to the wall surface on all four sides in order to prevent the whole infill panel, or large portions of it, from sliding and falling outward. Exterior wythes of multiwythe infills should be restrained from separating or peeling from the interior wythe (see Section C11.9.1.2A). Field and test observations indicate that infills constructed in tight contact with the surrounding frame can be considered to have adequate out-of-plane restraint. If a gap exists between the frame and the infill on any side, the gap must be filled with grout to provide tight contact, or out-of-plane restraint must be provided with other mechanical means.

Infills that are in tight contact with perimeter frame members develop arching mechanisms when subjected to out-of-plane loads. The out-of-plane capacity of an infill panel can be increased substantially through such an arching mechanism. However, formation of arching mechanisms requires the frame members to have substantial stiffness and strength to resist the thrust forces imparted on them by the arching infill. In general, if the infills are continuous—that is, adjacent bays and story levels are also infilled—the boundary conditions required for arch-mechanism formation may be assumed to be satisfied. For infills with open adjacent bays or story levels, the strength and stiffness of the frame members must be checked to confirm their adequacy.

A lower bound estimate of the transverse infill strength is given by Equation 7-21. The equation is a simplified version of one derived by Angel et al. (1994). Flexibility of beam or column members is included in the expression if their $E_{fe}I_f$ values exceed the minimum of 3.6 x $10^6$ lb-in.$^2$ as specified in the previous section. According to the theory, frame members with stiffnesses as low as this value should lower transverse strength by as much as 0.6. The lower bound strength equation also includes a reduction of 76% for an estimated amount of in-plane cracking for the most slender panel permitted. In this case, in-plane deflections equal to 50% more than those at initial cracking have been assumed.

### C7.5.3.3 Deformation Acceptability Criteria

Because out-of-plane infills are local elements spanning across individual stories and bays, limit states are expressed in terms of lateral deflection across their story height or length between columns.

The Immediate Occupancy Performance Level is not necessarily related to initial cracking of a wall. Some cracking can be tolerated for typical occupancy conditions.

Life Safety is related to extensive cracking of the infill panel. If arching action can be developed, the lateral story drift ratio of the most slender panel permitted ($h_{inf}/t_{inf}$ = 20) according to Equation 7-20 will be 2.8%, which is just less than the limit of 3.0% given for Life Safety. Thus, all infills that can develop arching mechanisms can meet this required Performance Level, provided that their strength will be sufficient to resist inertial forces.

## C7.6 Anchorage to Masonry Walls

According to Section 8.3.12 of BSSC (1995), the pullout strength of anchors is governed by the strength of the steel or the anchorage strength of the masonry. When practical, sufficient anchorage should be provided so that the anchor steel will yield, and a brittle pullout failure will be avoided. A ductile anchor will help insure a uniform distribution of force to individual anchors in the case that one or a few anchors are overloaded.

Ductility of an anchor will not significantly influence global ductility of a structural system, because plastic anchor extensions will be quite short relative to inelastic deformations of structural members. Anchors should be considered as force-controlled components, to ensure that the forces delivered to them by adjacent members will be resisted without inelastic straining or pullout of the anchor.

The effective embedment length is the length used to estimate the projected area of a pullout cone of masonry. Per Section 8.3.12 of BSSC (1995), this length is the length of embedment normal to the wall surface to the bearing surface of an anchor plate or head of an anchor bolt, or within one bar diameter from a hooked end.

When the embedment length is less than the minimum length prescribed by Section 8.3.12.1.4 of BSSC (1995), the pullout strength cannot be estimated reliably.

Shear strength of anchorages with edge distances less than 12 bolt diameters can be reduced by linear interpolation to zero at an embedment distance of one inch (25.4 mm).

## C7.7 Masonry Foundation Elements

No commentary is provided for this section.

## C7.8 Definitions

All definitions for Chapter 7 are given in the *Guidelines*.

## C7.9 Symbols

$A_v$ Shear area of wall or pier, in.$^2$

$E_{me}$ Expected elastic modulus of masonry in compression as determined in Section 7.3.2.2, psi

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

$I_e$ Effective moment of inertia of reinforced wall or pier per Equation C7-8, in.$^4$

$I_g$ Moment of inertia for uncracked, gross section, in.$^4$

$I_f$ Moment of inertia of beam or column member, in.$^4$

$L$ Length of wall or pier, in.

$L_{inf}$ Length of infill panel, in.

$M_u$ Moment at crushing of masonry, lb-in.

$M_y$ Moment at yield of reinforcement, lb-in.

$Q_{CE}$ Lower-bound estimate of the strength of a component or element at the deformation level under consideration

$Q_{UD}$ Deformation-controlled design action

$R_1$ Out-of-plane infill strength reduction factor to account for in-plane damage

$a$ Width of equivalent strut representing in-plane infill panel, in.

$d$ Effective depth of reinforced section, in.

$f_a$ Expected amount of vertical compressive stress based on load combinations given in Equations 3-1 and 3-2, psi

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

$f_{te}$ Expected masonry tensile strength as determined per Section 7.3.2.3, psi

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

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

$k$ Lateral stiffness of shear wall or pier, lb-in.

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

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

$l_p$ Length of plastic hinge for reinforced masonry wall or pier, in.

$m$ Factor to account for inelastic deformation capacity used in Equation 3-18

$q_{cr}$ Uniform transverse load when flexural cracking commences

$v_t$ Wall shear strength, 50th percentile, psi

$\Delta_{cr}$ In-plane deflection of infill panel at first cracking, in.

$\Delta_{inf}$ Out-of-plane deflection of infill panel at midspan, in.

$\varepsilon_{mu}$ Crushing strain of masonry

$\mu_{\Delta}$ Displacement ductility for reinforced wall or pier section

$\mu_{\phi}$ Curvature ductility for reinforced wall or pier section

$\theta_b$ Angle between lower edge of compression strut and beam as shown in Figure C7-5, radians

$\theta_c$ Angle between lower edge of compression strut and beam as shown in Figure C7-4, radians

$\phi_y$ Curvature at initial yield of reinforcement, 1/in.

$\phi_u$ Curvature at crushing of masonry, 1/in.

## C7.10 References

ABK, 1981, *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Wall Testing, Out-of-Plane,* Topical Report 04.

Abrams, D. P., 1992, “Strength and Behavior of Unreinforced Masonry Elements,” *Proceedings of Tenth World Conference on Earthquake Engineering,* Balkema Publishers, Rotterdam, The Netherlands, pp. 3475–3480.

Abrams, D. P., editor, 1994, *Proceedings of the NCEER Workshop on Seismic Response of Masonry Infills,* Report No. NCEER-94-0004, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Abrams, D. P., and Matthys, J., 1991, “Present and Future Techniques for Nondestructive Evaluation of Masonry Structures,” *Journal of The Masonry Society*, Boulder, Colorado, Vol. 10, No. 1, pp. 22–30.

Abrams, D. P., and Shah, N., 1992, *Cyclic Load Testing of Unreinforced Masonry Walls*, Advanced Construction Technology Center Report #92-26-10, College of Engineering, University of Illinois at Urbana, Illinois.

Abrams, D. P., Angel, R., and Uzarski, J., 1993, “Transverse Strength of Damaged URM Infills,” *Journal of The Masonry Society*, Boulder, Colorado, Vol. 12, No. 1, pp. 45–52.

Agbabian, M. S., Adham, S., Masri, S., Avanessian, V., and Traina, I., 1989, *Out-of-Plane Dynamic Testing of Concrete Masonry Walls*, TCCMAR Report No. 3.02(b1), University of Southern California, Los Angeles, California.

Angel, R., Abrams, D. P., Shapiro, D., Uzarski, J., and Webster, M., 1994, *Behavior of Reinforced Concrete Frames with Masonry Infills*, Structural Research Series No. 589, UILU-ENG-94-2005, University of Illinois at Urbana, Illinois.

ASTM, latest edition, Standards with the following numbers: C597, C1072, C1197, E518, E519, American Society for Testing Materials, Philadelphia, Pennsylvania.

Atkinson, R. H., Amadei, B. P., Saeb, S., and Sture, S., 1989, “Response of Masonry Bed Joints in Direct Shear,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 115, No. 9, pp. 2276–2296.

Blondet, M., and Mayes, R. L., 1991, *The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes*, TCCMAR Report No. 3.2(b2), Computech Engineering Services, Inc., Berkeley, California.

BSSC, 1992, *NEHRP Handbook for the Seismic Evaluation of Existing Building*s, developed by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report No. FEMA 178), Washington, D.C.

BSSC, 1995, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings, 1994 Edition, Part 1: Provisions and Part 2: Commentary*, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report Nos. FEMA 222A and 223A), Washington, D.C.

Calvi, G. M., 1988, "Correlation between Ultrasonic and Load Tests on Old Masonry Specimens," *Proceedings of Eighth International Brick/Block Masonry Conference*, Elsevier Applied Science, Essex, England, pp. 1665–1672.

Calvi, G. M., Kingsley, G. R., and Magenes, G., 1996, "Testing of Masonry Structures for Seismic Assessment," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California, Vol. 12, No. 1, pp. 145–162.

Costley, A. C., and Abrams, D. P., 1995, *Dynamic Response of URM Buildings with Flexible Diaphragms*, Structural Research Series No. 605, University of Illinois at Urbana, Illinois.

Curtin, W. G., Shaw, G., and Beck, J. K., 1988, *Design of Reinforced and Prestressed Masonry*, Thomas Telford Publishers, London, United Kingdom.

Durrani, A. J., and Luo, Y. H., 1994, "Seismic Retrofit of Flat-Slab Buildings with Masonry Infills," *Proceedings of NCEER Workshop on Seismic Response of Masonry Infills*, Technical Report NCEER-94-0004, San Francisco, California, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Epperson, G. S., and Abrams, D. P., 1989, *Nondestructive Evaluation of Masonry Buildings*, Advanced Construction Technology Center Report No. 89-26-03, College of Engineering, University of Illinois at Urbana, Illinois.

Epperson, G. S., and Abrams, D. P., 1992, "Evaluating Lateral Strength of Existing Unreinforced Brick Masonry Piers in the Laboratory," *Journal of The Masonry Society*, Boulder, Colorado, Vol. 10, No. 2, pp. 86–93.

Ewing, R. D., El-Mustapha, A. M., and Kariotis, J. C., 1987, *FEM/I: A Finite Element Computer Program for the Nonlinear Static Analysis of Reinforced Masonry Building Components*, TCCMAR Report No. 2.2-1, R. D. Ewing and Associates, Palos Verdes, California.

Flanagan, R. D., Bennett, R. M., and Barclay, G. A., 1993, "In-Plane Behavior and Strength of Structural Clay Tile Infilled Frames," *Proceedings of the Sixth North American Masonry Conference*, Drexel University, Philadelphia, Pennsylvania, pp. 371–382.

Flanagan, R. D., Tenbus, M. A., and Bennett, R. M., 1994, "Numerical Modeling of Clay-Tile Infills," *Proceedings of NCEER Workshop on Seismic Response of Masonry Infills*, Technical Report NCEER-94-0004, San Francisco, California, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Gergely, P., White, R. N., and Mosalam, K. M., 1994, "Evaluation and Modeling of Infilled Frames," *Proceedings of NCEER Workshop on Seismic Response of Masonry Infills*, Technical Report NCEER-94-0004, San Francisco, California, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Hamburger, R. O., 1993, "Methodology for Seismic Capacity Evaluation of Steel-Frame Buildings with Infill Unreinforced Masonry," *Proceedings of 1993 National Earthquake Conference*, Central U.S. Earthquake Consortium, Memphis, Tennessee, Vol. II, pp. 173–191.

Hamid, A. A., Abboud, B. E., Farah, M. W., Hatem, M. K., and Harris, H. G., 1989, *Response of Reinforced Block Masonry Walls to Out-of-Plane Static Loads*, TCCMAR Report No. 3.2(a), Department of Civil and Environmental Engineering, Drexel University, Philadelphia, Pennsylvania.

He, L., and Priestley, M.J.N., 1992, *Seismic Behavior of Flanged Masonry Shear Walls*, TCCMAR Report No. 4.1-2, Department of Applied Mechanics and Engineering Sciences, University of California at San Diego, California.

Holmes, M., 1961, “Steel Frames with Brickwork and Concrete Infilling,” *Proceedings of Institution of Civil Engineers*, London, United Kingdom, Vol. 19, pp. 473–478.

Holmes, M., 1963, “Combined Loading on Infilled Frames,” *Proceedings of Institution of Civil Engineers*, London, United Kingdom, Vol. 25, pp. 31–38.

ICBO, 1994, *Uniform Code for Building Conservation*, International Conference of Building Officials, Whittier, California.

Kariotis, J., Guh, T. J., Hill, J. A., and Youssef, N.F.G., 1994, “Simulation of the Recorded Response of Unreinforced (URM) Infill Buildings,” *Proceedings of NCEER Workshop on Seismic Response of Masonry Infills*, Technical Report NCEER-94-0004, San Francisco, California, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Kariotis, J., and Ngheim, D., 1995, “In-Situ Determination of Compressive Stress-Strain Relationship of Multi-Wythe Brick Masonry,” *Proceedings of ASCE Structures Congress*, American Society of Civil Engineers, New York, New York, Vol. 2, pp. 1421-1426.

Kingsley, G. R., Noland, J. L., and Atkinson, R. H., 1987, “Nondestructive Evaluation of Masonry Structures Using Sonic and Ultrasonic Pulse Velocity Techniques,” *Proceedings of Fourth North American Masonry Conference*, The Masonry Society, Boulder, Colorado.

Kingsley, G. R., 1995, “Evaluation and Retrofit of Unreinforced Masonry Buildings,” *Proceedings of the Third National Concrete and Masonry Engineering Conference*, San Francisco, California, pp. 709–728.

Klingner, R. E., editor, 1994, *Performance of Masonry Structures in the Northridge Earthquake of January 17, 1994*, The Masonry Society, Boulder, Colorado.

Klingner, R. E., and Bertero, V. V., 1976, *Infilled Frames in Earthquake Resistant Construction*, Report No. EERC 76-32, Earthquake Engineering Research Center, University of California at Berkeley, California.

Leiva, G., and Klingner, R. E., 1991, *In-Plane Seismic Resistance of Two-Story Concrete Masonry Shear Walls with Openings*, TCCMAR Report No. 3.1(c)-2, Department of Civil Engineering, University of Texas at Austin, Texas.

Lenczner, D., 1986, “Creep and Prestress Losses in Brick Masonry,” *The Structural Engineer,* Institution of Structural Engineers, United Kingdom, Vol. 64B, No. 3, pp. 57–62.

Lotfi, H. R., and Shing, P. B., 1994, “Interface Model Applied to Fracture of Masonry Structures,” *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, Vol. 120, No. 1.

Mainstone, R. J., and Weeks, G. A., 1971, “The Influence of a Bounding Frame on the Racking Stiffness and Strength of Brick Walls,” *Current Paper CP 3/72*, Building Research Station, Garston, United Kingdom, also published in *Proceedings of Second International Brick Masonry Conference*, pp. 165–171.

Mainstone, R. J., 1971, “On the Stiffnesses and Strengths of Infilled Frames,” *Current Paper CP 2/72*, Building Research Station, Garston, United Kingdom, reprinted from *Proceedings of Institution of Civil Engineers*, 1971 Supplement (iv), Paper 7360 S, pp. 57–90.

Mainstone, R. J., 1974, “Supplementary Note on the Stiffnesses and Strengths of Infilled Frames,” *Current Paper CP 13/74*, Building Research Station, Garston, United Kingdom.

Masonry Standards Joint Committee (MSJC), 1995a, *Building Code Requirements for Masonry Structures*, ACI 530-95/ASCE 5-95/TMS 402-95, American Concrete Institute, Detroit, Michigan, American Society of Civil Engineers, New York, New York, and The Masonry Society, Boulder, Colorado.

Masonry Standards Joint Committee (MSJC), 1995b, *Specification for Masonry Structures*, ACI 530.1-95/ ASCE 6-95/TMS 602-95, American Concrete Institute, Detroit, Michigan, American Society of Civil Engineers, New York, New York, and The Masonry Society, Boulder, Colorado.

Mehrabi, A. B., and Shing, P. B., 1994, “Performance of Masonry-Infilled R/C Frames Under In-Plane Lateral Loads: Analytical Modeling,” *Proceedings of NCEER Workshop on Seismic Response of Masonry Infills*, San

Francisco, California, Technical Report NCEER-94-0004, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York.

Merryman, K. M., Leiva, G., Antrobus, N., and Klingner, R. E., 1990, *In-Plane Seismic Resistance of Two-Story Concrete Masonry Coupled Shear Walls*, TCCMAR Report No. 3.1(c)-1, Department of Civil Engineering, University of Texas at Austin, Texas.

NBS, *Evaluation of Structural Properties of Masonry in Existing Buildings*, Building Science Series 62, National Bureau of Standards (now National Institute for Science and Technology), U.S. Department of Commerce, Washington, D.C.

Noland, J. L., Atkinson, R. H., and Kingsley, G. R., 1987, "Nondestructive Methods for Evaluating Masonry Structures," *Proceedings of International Conference on Structural Faults and Repair*, London, United Kingdom.

Paulay, T., and Priestley, M.J.N., 1992, *Seismic Design of Reinforced Concrete and Masonry Buildings*, Wiley Interscience, New York, New York.

Paulson, T. J. and Abrams, D. P., 1990, *Measured Inelastic Response of Reinforced Masonry Building Structures to Earthquake Motions*, TCCMAR Report No. 7.1-1, Structural Research Series No. 555, University of Illinois at Urbana-Champaign, Illinois.

Plummer, H. C. and Blume, J. A., 1953, *Reinforced Brick Masonry and Lateral Force Design*, Structural Clay Products Institute (now Brick Institute of America), Reston, VA.

Polyakov, S. V., 1957, "Masonry in Framed Buildings: An Investigation into the Strength and Stiffness of Masonry Infilling" (English translation), Moscow, Russia.

Polyakov, S. V., 1960, "On the Interaction Between Masonry Filler Walls and Enclosing Frame When Loaded in the Plane of the Wall," *Translations in Earthquake Engineering*, Earthquake Engineering Research Institute, Oakland, California, pp. 36–42.

Priestley, M. J. N., and Hart, G. C., 1989, *Design Recommendations for the Period of Vibration of Masonry Wall Buildings*, Report No. SSRP-89/05, Department of Applied Mechanics and Engineering Sciences, University of California at San Diego, California.

Sansalone, M., and Carino, N., 1988, "Impact-Echo Method, Detecting Honeycombing, the Depth of Surface Opening Cracks, and Ungrouted Tendons," *Concrete International*, American Concrete Institute, Detroit, Michigan, p. 38.

Seible, F., Hegemier, G. A., Priestley, M.J.N., Kingsley, G. R., Kurkchubasche, A. G., Igarashi, A., and Weeks, J. S., 1994, *The US-TCCMAR Full-Scale Five-Story Masonry Research Building Test, Part I - Executive Summary*, TCCMAR Report No. 9.4-1, Department of Applied Mechanics and Engineering Sciences, University of California at San Diego, California.

Shing, P. B., Noland, J. L., Spaeh, H. P., Klamerus, E. W., and Schuller, M. P., 1991, *Response of Single-Story Reinforced Masonry Shear Walls to In-Plane Lateral Loads*, TCCMAR Report No. 3.1(a)-2, Department of Civil, Environmental and Architectural Engineering, University of Colorado at Boulder, Colorado.

Stafford Smith, B., 1962, "Lateral Stiffness of Infilled Frames," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, pp. 183–199.

Stafford Smith, B., 1966, "Behavior of Square Infilled Frames," *Journal of the Structural Engineering Division*, American Society of Civil Engineers, New York, New York, pp. 381–403.

Stafford Smith, B., 1968, "Model Test Results of Vertical and Horizontal Loading of Infilled Frames," *ACI Structural Journal*, American Concrete Institute, Detroit, Michigan, pp. 618–624.

Stafford Smith, B., and Carter, C., 1969, "A Method of Analysis for Infilled Frames," *Proceedings of the Institution of Civil Engineers*, London, United Kingdom, Vol. 44, pp. 31–48.

Tena-Colunga, A., and Abrams, D. P., 1992, *Response of an Unreinforced Masonry Building During the Loma Prieta Earthquake*, Structural Research Series No. 576, University of Illinois at Urbana-Champaign, Illinois.

Turnsek, V., and Sheppard, P., 1980, “The Shear and Flexural Resistance of Masonry Walls,” *Proceedings of the International Research Conference on Earthquake Engineering*, Skopje, Yugoslavia.

Xu, W., and Abrams, D. P., 1992, *Evaluation of Lateral Strength and Deflection for Cracked Unreinforced Masonry Walls, Advanced Construction Technology Center Report #92-26-11*, College of Engineering, University of Illinois at Urbana, Illinois.