Chapter 11 Commentary

MASONRY STRUCTURE DESIGN REQUIREMENTS

11.1 GENERAL:

11.1.1 Scope: The provisions of Chapter 11 govern design and construction of all types of masonry. Quality assurance is covered with a reference to Chapter 3. Reinforced and plain (unreinforced) masonry elements that are part of the basic structural system and those that are not part of the basic structural system are included.

11.1.2 Reference Documents: Design and construction standards cited in Chapter 11 are listed in Sec. 11.1.2. The materials standards are specifically listed to include only those materials permitted by the provisions. The listing includes the document's designation, the year of the edition and the title of the document.

11.1.3 Definitions: Terms used in the provisions which have a specific meaning which differs from the dictionary definition are defined in Sec. 11.1.3. All other terms are defined by the dictionary.

11.1.4 Notations: Notations used in the provisions are defined in Sec. 11.1.4. English units of measure are stated followed by the metric unit in parenthesis for each term.

11.2 CONSTRUCTION REQUIREMENTS:

11.2.1 General: ACI 530.1 is a standard specification prepared under consensus procedures. It was developed by members representing construction, design, materials, and research of masonry structures. The document is intended to be incorporated into contract documents used to construct masonry structures.

This standard specification was developed to be used in conjunction with *Building Code Requirements for Masonry structures*, ACI 530. Appropriate standards for materials and test methods are referenced. In addition to a general section, there are sections on masonry, reinforcement and metal accessories, and grout.

The materials listed in ACI 530.1 have been restricted in order to obtain more predictable behavior and better performance required for strength design. Construction provisions found in Chapter 11 override those found in ACI 530.1.

11.2.2 Quality Assurance: See Chapter 3 of the *Provisions* and *Commentary*. Quality assurance requirements for masonry structures include testing of masonry *components* (mortar, grout, and units) or testing of masonry assemblages. Industry guidelines for materials testing are listed below.

1. Brick Institute of America, 11490 Commerce Park Drive, Reston, Virginia 22091, *Technical Notes on Brick Construction*:

No. 39 Revised, "Testing for Engineered Brick Masonry: Brick, Mortar and Grout," January 1987.

No. 39A, "Testing for Engineered Brick Masonry: Determination of Allowable Design Stresses," December 1987.

No. 39B, "Testing for Engineering Brick Masonry: Quality Assurance," March 1988.

2. National Concrete Masonry Association, 2302 Horse Pen Road, Herndon, Virginia 22071-3499:

   TEK 22A, *Prism Testing for Engineered Concrete Masonry*, 1979.

   TEK 107, *Laboratory and Field Testing of Mortar and Grout*, 1979.

   TEK 108, *Testing Concrete Masonry Assemblages*, 1979.

Industry guidelines for field inspection are listed below.

1. Brick Institute of America, 11490 Commerce Park Drive, Reston, Virginia 22091, *Technical Notes on Brick Construction*:

   No. 17C, "Reinforced Brick Masonry: Inspectors' Guide," May 1986.

2. National Concrete Masonry Association, 2302 Horse Pen Road, Herndon, Virginia 22071-3499:

   TEK 65, *Field Inspection of Engineered Concrete Masonry*, 1975.

   TEK 132, *Inspector's Guide for Concrete Masonry Construction*, 1983.

11.3 GENERAL REQUIREMENTS:

11.3.1 Scope: This chapter offers three different methods for designing masonry structures. Any method, used within the limitations imposed, provides acceptable masonry construction with acceptable seismic resistance characteristics.

11.3.2 Empirical Masonry Design: Empirical design methods are based on the successful performance of masonry buildings. Prescriptive requirements and limited exposure to loads are necessary to ensure compliance.

The design process results in sizes and proportions of masonry elements using minimum thicknesses and maximum spans. Although rudimentary stress calculations are made, empirical masonry design does not require a complete structural analysis.

11.3.3 Plain (Unreinforced) Masonry Design: Design methods for plain masonry, often referred to as unreinforced masonry. The procedures utilize working stress design requirements using principles of mechanics.

11.3.4 Reinforced Masonry Design: Reinforcing steel complements the high compressive strength of masonry with high tensile strength. Increased load-carrying capacity and greater ductility result from the use of reinforcing steel.

11.3.5 - 11.3.9 Seismic Design Categories A through F: Any type of masonry shear wall is permitted in Seismic Design Categories A and B. Detailed plain masonry shear walls or intermediate reinforced masonry shear walls are required for Seismic Design Category C. Special reinforced masonry shear walls are required for Seismic Design Categories D, E, or F. Minimum requirements for each type of masonry shear wall are given in Sec. 11.11. These requirements are consistent with intended inelastic deformation capacities that are the bases for the R, Ω, and C_d factors given in Table 5.2.2. Additional requirements for construction of masonry elements other than shear walls are given for each Seismic Design Category in Sec. 11.3.5 through 11.3.9

11.3.6 Seismic Design Category B: The use of empirical masonry design, Sec. 11.3.2, for the lateral load resisting system is not appropriate for Seismic Design Category B. Masonry walls that are not part of the lateral load resisting system may be designed by the empirical method.

11.3.10: Properties of Materials:

11.3.10.1 Steel Reinforcement Modulus of Elasticity: The given modulus of elasticity of steel reinforcement is taken from previous codes and is consistent with established design values. Design may be based on tested values of modulus of elasticity; however, these tests are rarely performed because it is impractical to test materials to be used in the construction at the time when the project is being designed.

11.3.10.2 Masonry Modulus of Elasticity: Modulus of elasticity of masonry is used in determining stiffness of structural *components* prior to cracking. Therefore, the modulus is taken from the elastic portion of the stress strain curve. The modulus of elasticity of masonry is not clearly related to any property of mortar, unit, grout or prism h/t, but is influenced by all of these. TS5 concluded it was best to relate the value of E_m to the specified compressive strength of masonry. This is because f_m' is also influenced by these parameters. The 750 multiplier is used rather than lower multipliers reported (Wolde-Tinsae, 1993) since the actual compressive strength of masonry must exceed the specified compressive strength.

11.3.10.4 Masonry Compressive Strength: Research has been performed on structural masonry *components* having a compressive strength in the range of 1,500 to 6,000 psi (10 to 41 MPa). Design criteria are based on these research results. Design values therefore are limited to compressive strengths in the range of 1,500 to 4,000 psi (10 to 28 MPa) for concrete masonry and 1,500 to 6,000 psi (10 to 41 MPa) for clay masonry.

11.3.10.5 Modulus of Rupture: Modulus of rupture values in Table 11.3.10 are based on allowable working stress values for flexural tension multiplied by 2.0 to approximate the lower limit of strength values. See the Commentary to ACI 530 for discussion. Stack bond masonry has historically been assumed to have no flexural bond strength across the head joints; thus, the grout area alone is used.

11.3.10.6 Reinforcement Strength: Research conducted on reinforced masonry *components* used Grade 60 reinforcement. To be consistent with laboratory documented performance, design is based on a steel yield strength that does not exceed 60,000 psi (413 MPa).

11.3.11 Section Properties: Section properties of masonry members are available in masonry design publications. Design is based on specified dimension. Actual dimensions may vary within the tolerance range given in the construction requirement (i.e., ACI 530.1). The strength reduction factors are based in part on an anticipated variation in the specified (design) dimensions.

11.3.12 Headed and Bent-Bar Anchor Bolts: This section covers cast-in-place headed anchor bolts and bent-bar anchors (J- or L-bolts) in grout. General background information on this topic is given in CEB, 1995.

The tensile capacity of a headed anchor bolt is governed by yield and fracture of the anchor steel or by breakout of a roughly conical volume of masonry starting at the anchor head and having a fracture surface oriented at 45 degrees to the masonry surface. Steel capacity is calculated conventionally using the effective tensile stress area of the anchor (i.e., including the reduction in area of the anchor shank due to threads). Masonry breakout capacity is calculated using expressions adapted from concrete design, which use a simplified design model based on a stress of $4\sqrt{f_m'}$ uniformly distributed over the area of that right circular cone, projected onto the surface of the masonry. Reductions in breakout capacity due to nearby edges or adjacent anchors are computed in terms of reductions in those projected areas (Brown and Whitlock, 1983).

The tensile capacity of a bent-bar anchor bolt (J- or L-bolt) is governed by yield and fracture of the anchor steel, by tensile cone breakout of the masonry, or by straightening and pullout of the anchor from the masonry. Capacities corresponding to the first two failure modes are calculated as for headed anchor bolts. Pullout capacity is calculated as proposed by Shaikh, 1996. Possible contributions to tensile pullout capacity due to friction are neglected.

The tensile breakout capacity of a headed anchor is usually much greater than the pullout capacity of a J- or L-bolt. the designer is encouraged to use headed anchors when anchor tensile capacity is critical.

The shear capacity of a headed or a bent-bar anchor bolt is governed by yield and fracture of the anchor steel or by masonry shear breakout. Steel capacity is calculated conventionally using the effective tensile stress area (i.e., threads are conservatively assumed to lie in the critical shear plane). Shear breakout capacity is calculated as proposed ;by Brown and Whitlock, 1983.

Under static shear loading, bent-bar anchor bolts (J- or L-bolts) do not exhibit straightening and pullout. Under reversed cyclic shear, however, available research suggests that straightening and pullout may occur. Headed anchor bolts are recommended for such applications (Malik et al., 1982).

11.4 DETAILS OF REINFORCEMENT:

11.4.5 Development of Reinforcement:

11.4.5.2 Development of Reinforcing Bars & Wires in Tension: In order to have ductile behavior of a masonry member subjected to seismic loads, the strength of the bar or wire must be developed by embedment. The development length given by Eq. 11.4.5.2 is based on an analysis of the results of multiple independent research efforts (NCMA, U.S. Army Corps of Engineers by Atkinson-Noven and Associates, and Washington State University) investigating the performance of lap splices and the requirements for development of reinforcement.

Using the compiled data from these studies, numerous multiple regression analyses were performed to identify the parameters having a significant effect on lap splice and development length. The most important parameters are compressive strength of the masonry assemblage (prism strength), diameter of the reinforcing bar, spacing of the bars, and cover. The best-fit equation developed in the regression analyses was simplified for design purposes to Eq. 11.4.5.2 while retaining all the essential parameters.

The lap lengths required by the proposed equation provide a capacity in excess of 1.25 f_y. (Note that an additional factor to account for material variability, construction defects, and other design uncertainties is included by use of the phi factor in the strength design formula.)

11.5 STRENGTH AND DEFORMATION REQUIREMENTS:

11.5.3 Design Strength: The design strength of a member and its connections is calculated by engineering principles and materials strength and yield values. This calculated strength is the nominal strength of the member. The nominal strength is less than the expected or mean strength because minimum guaranteed values or specified strengths are used for the calculations of nominal strength. A strength reduction factor, ϕ, is used to reduce the nominal strength to a design strength. The strength reduction factor, ϕ, is a variable that is dependent on the material and material behavior. Flexural strength of reinforced members is reduced less by the ϕ factor than is shear strength. Exceeding of the flexural strength of a reinforced member causes yielding of the reinforcement but not strength degradation. Exceeding of the shear strength results in a strength degradation.

Flexure Without Axial Load: The strength reduction factor for reinforced masonry is greater than for plain masonry because plain masonry after cracking lacks ductile performance.

Axial Load and Axial Load with Flexure: If the axial load results in balanced strain conditions (flexure produces strain in the reinforcement equal to the yield strain and strain in the masonry equal to the maximum usable strain, ϵ_{mu}) and the flexural reinforcement is minimal, an increase in flexural moment can cause compressive stresses in excess of the compressive strength. The failure will not be ductile; therefore, the strength reduction factor is more severe. Linear interpolation of the strength reduction factor is allowed since the required axial strength due to factored load, P_u, decreases from the axial load resulting in balanced strain conditions to zero, so as to make the transition linear from axial load with flexure to flexure without axial load.

The strength reduction factor for the vertical members of wall frames is more restrictive than for shear walls or coupled shear walls. The strength reduction factor for the vertical members of wall frames does not have a linear variation to its value. When $P_u/A_n f'_m$ is equal to 0.1, the strength reduction factor will be equal to 0.65.

The strength reduction factor for plain masonry members is unchanged from that factor that is applied for flexure only. Axial load increases the flexural capacity of plain masonry but does not significantly change its lack of ductility.

Shear: Strength reduction factors for calculation of design shear strength are commonly more severe than those factors used for calculation of design flexural strength. This concept is partially supported by the wider variance of shear capacities that have been obtained from experimental testing. The variance of the results of each experiment from the body of data is due not only to the variability of the masonry materials, the test apparatus and test methods, and the shear strength parameters tested but also to the greater sensitivity of shear resistance mechanisms to those factors.

Bearing: Exceeding of the bearing capacity causes crushing and spalling of bearing surfaces. The strength reduction factors given are those established for elements that have strength degradation.

11.5.4 Deformation Requirements: Stiffness of a structural element is as important or more important than strength. Stiffness is critical for serviceability and control of displacements. Drift of an element is the movement of one story of the building relative to the adjacent stories or the displacement of the shear wall relative to its fixed base. Drift of the top level of a shear wall is affected by foundation flexibility but the structural stresses and strains in the wall would not be increased by foundation flexibility.

The product of the effective moment of inertia, I, and the effective modulus of elasticity, E, is usually used as a variable for the calculation of the deformation of reinforced elements. The variability in I is caused by tensile cracking of the masonry cross section. If tensile cracking is not acceptable, as for plain masonry, I has a single value and the compressive modulus of elasticity and the moment of inertia of the gross cross section is used for the calculation of deformation.

If tensile cracking in anticipated, such as for reinforced masonry, the effective I at every cross section of the wall or beam is dependent on the curvature of the cross section and the shear deformation of each increment of the member length. Several nonlinear finite element programs have the capability of determining the stiffness degradation of reinforced masonry elements, but the effective stiffness, I, can be determined by use of Eq. 11.5.4.3.

The cracking moment is calculated using the section modulus of the gross section of wall times the modulus of rupture of masonry, f_r. The moment of inertia of the cracked section is calculated about the neutral axis of the section, using the masonry properties, and transforming the reinforcement into equivalent masonry areas by use of the ratio of the compressive modulus of steel and masonry. The cracked moment of inertia, I_{cr}, and the compressive modulus of masonry, E_m, is used to calculate the effective moment of inertia, I_{eff}.

Eq. 11.5.4.3 has been used as a means of providing a transition in stiffness between gross moment of inertia and a totally cracked section. Abboud (1987), Abboud and Hamid (1987), Abboud et al. (1990 and 1993), Hamid et al. (1989), and Horton and Tadros (1990) give additional insight and behavior for computing deflection for masonry *components*.

11.6 FLEXURE AND AXIAL LOADS:

11.6.2 Design Requirements of Reinforced Masonry Members: The design principles listed are those that traditionally have been used for reinforced masonry members. The theory used for design of normally proportioned flexural members has limited applicability to deep flexural members. Shear warping of the cross section and a combination of diagonal tension stress and flexural tension stresses in the body of the deep flexural members require that deep beam theory be used for members that exceed the specified limits of span to depth ratio.

11.6.2.2: Longitudinal reinforcement in flexural members is limited to a maximum amount to ensure that masonry compressive strains will not exceed ultimate values~~.~~ -- in other words, that the compressive zone of the member will not crush before the tensile reinforcement develops the inelastic strain consistent with the maximum drift limits of Provisions Table 5.2.8.

For all masonry *components* other than walls bending in the out-of-plane sense and masonry structures expected to remain essentially elastic, maximum reinforcement is limited in accordance with a prescribed strain distribution based on a tensile strain equal to five times the yield strain for the reinforcing bar closest to the edge of the member, and a maximum masonry compressive strain equal to 0.0025 for concrete masonry or 0.0035 for clay-unit masonry. By limiting longitudinal reinforcement in this manner, inelastic curvature capacity is easily depicted as the slope of the strain distribution.

Because axial force is implicitly considered in the determination of maximum longitudinal reinforcement, inelastic curvature capacity can be relied on no matter what the level of axial compressive force. Thus, the capacity reduction factors, ϕ, for axial load and flexure can be the same as for flexure alone. Also, confinement reinforcement is not required because the maximum masonry compressive strain will be less than ultimate values.

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

The numerical limits in the required provisions can be developed consistently from several approaches.

For structures expected to respond inelastically, one approach for the in-plane limits is based on the design model of a cantilever wall with a flexural hinge at the base. Neglecting elastic deformations of the wall and considering only the concentrated inelastic rotation at the hinge, the required inelastic curvature at the base of the wall is geometrically related to the maximum drift limits of Provisions Table 5.2.8. Assuming for simplicity that the length of the plastic hinge is equal to the plan length of the wall, the maximum drift ratio must equal the summation of the maximum usable compressive strain in the masonry plus the expected strain in the tensile reinforcement. Using a maximum usable compressive strain in the masonry between 0.0025 and 0.0035, the expected strain in the tensile reinforcement is about 4 times yield. The expected strain in the tensile reinforcement must somewhat exceed this, however, because the inelastic rotation is not concentrated at a single point, and because the inelastic hinge length can be less than the plan length of the wall. The required expected strain is therefore set at 5 times yield.

For structures expected to respond inelastically, the masonry compressive force is estimated using a rectangular stress block defined with parameters based on recent research done with the Technical Coordinating Committee for Masonry Research (TCCMaR).

For walls bending out of plane, the limit on maximum reinforcement is relaxed by considering a strain distribution based on 1.3 times the yield strain for the reinforcing bar closest to the member edge. This limiting strain distribution is less severe than that adopted for in plane bending. It is based on research done by Blondet and Mayes (1991).

For structures intended to undergo significant inelastic response, the Provisions are a technically sound way of achieving the design objective of inelastic deformation capacity. They are, however, unnecessarily restrictive for those structures not required to undergo significant inelastic deformation under the design earthquake.

Because unreinforced masonry structures, which have traditionally been regarded as non-ductile, are assigned an R value of 1.5, that value was taken as corresponding to essentially elastic response. Structures designed using an R value of 1.5 would be expected to reach but not exceed a critical strain condition corresponding to the development of yield strain in the extreme tensile reinforcement, and the development of the maximum useful compressive strain in the extreme compressive fiber of the masonry.

Because of the possibility of overstrength, yield strain was taken as 1.25 f_y. To allow a prudent margin of safety against limited inelastic behavior, the maximum strain in the tensile reinforcement was increased from 1.25 ε_y to 2 ε_y. The criterion for essentially elastic structures applies to both in- and out-of-plane flexure.

At the curvatures corresponding to yield strain or slightly past yield strain, the corresponding stress distribution in the masonry would be linear rather than an equivalent rectangle. Using that stress distribution, the axial load in the element, and the corresponding elastic stresses in tensile and compressive reinforcement, the maximum reinforcement can be calculated.

Maximum reinforcement per the requirements of Sec. 11.6.2.2.1 for an in-plane wall with uniformly distributed vertical reinforcement can be derived using simple equilibrium concepts to give:

$$\rho_{\max} = \frac{0.64 f_m' \alpha - \dfrac{P_g}{bd}}{1.25 f_y (1 - \alpha) - 0.5 f_{s\,\max} \alpha} \qquad \text{(C11.6.2.2-1)}$$

where ρ_{max} is the total amount of vertical steel divided by *b* and *d*; *b* is the width of the section; *d* is the distance from the extreme compressive fiber to the location of the tensile vertical bar closest to the edge of the member; α is equal to the depth of the compression zone divided by the effective depth, *d*; P_g is equal to the unfactored gravity compressive force; f_y is the specified yield stress of the reinforcement, and $f_{s\,max}$ is the maximum compressive stress in the vertical reinforcement.

Similarly, maximum reinforcement per the requirements of Sec. 11.6.2.2.2 for an out-of-plane wall with a single layer of vertical reinforcement centered on the wall section reduces to:

$$\rho_{\max} = \frac{0.64 f_m' \alpha - \dfrac{P_g}{bd}}{2.50 f_y} \qquad \text{(C11.6.2.2-2)}$$

where $\rho_{g\,\max}$ is the total amount of vertical steel divided by the gross area of the wall section.

For structures expected to undergo significant inelastic response, Equation C11.6.2.2-2 shows that for any maximum permitted reinforcement ratio, the Provisions directly limit the summation of forces from axial load and tensile reinforcement. Given prescriptive requirements for minimum reinforcement, the Provisions limit the axial load to a value that increases with the specified compressive strength of the masonry. This limitation may govern the maximum number of stories.

These maximum reinforcement requirements apply to all flexural members (walls, columns and beams) in all seismic design categories, for all structures expected to undergo significant inelastic response. Maximum reinforcement requirements for structures not expected to undergo significant elastic response, while similar in nature, impose less stringent limitations on maximum reinforcement.

For calibration purposes, maximum longitudinal reinforcement per Sec. 2108.2.3.3 of the 1997 Uniform Building Code is also plotted in Figures C11.6.2.2-1 and C11.6.2.2-2. The UBC criterion limits longitudinal reinforcement to no more than one-half of that resulting in a balanced condition where ultimate masonry compressive stress is equal to 0.003 and reinforcement is at its yield strain. A rectangular stress block is to be used with a stress equal to 0.85 times f'_m and a stress block depth equal to 0.85 times the compressed zone. No increase in the yield stress is specified by the UBC to account for increases due to higher expected strengths, strain hardening, or flexural cracking. The UBC criterion also considers axial force when limiting maximum reinforcement. However, in addition to gravity forces, axial forces due to earthquake effects times a load factor of 1.4 are also considered. Using the same procedure as used to derive the two former equations, the UBC criterion reduces to:

$$\rho_{g\ \max} = \frac{1}{2}\left[\frac{0.723 f'_m \alpha - \dfrac{P_u}{bd}}{2.00 f_y}\right] \qquad \text{(C11.6.2.2-3)}$$

where P_u is the factored axial load (1.0D + 1.0L +1.4E).

The UBC criterion results in a more restrictive limit on maximum reinforcement for axial compressive stress less than 381 psi for clay-unit masonry and 103 psi for concrete masonry (with the assumed values of f'_m). For axial compressive stresses above these values, the in-plane criterion per Sec. 11.6.2.2 results in a more restrictive limit on maximum reinforcement.

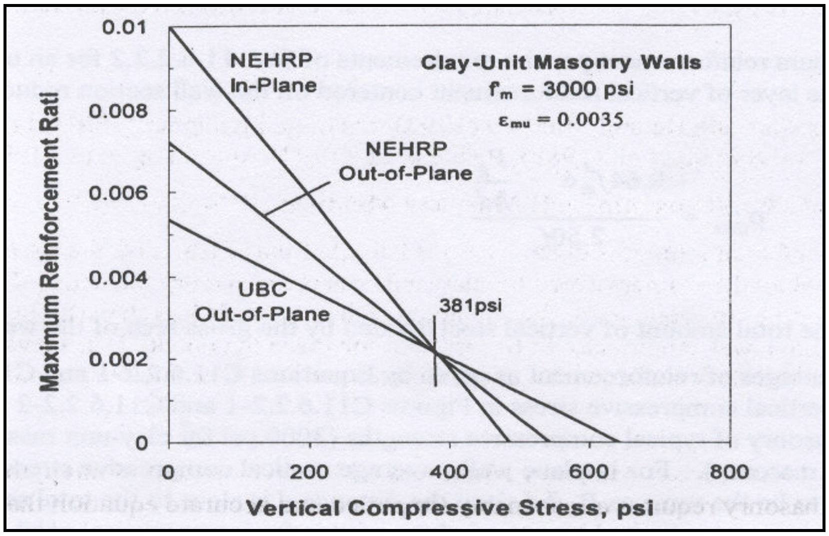

FIGURE C11.6.2.2-1 Maximum reinforcement for clay-unit masonry walls.

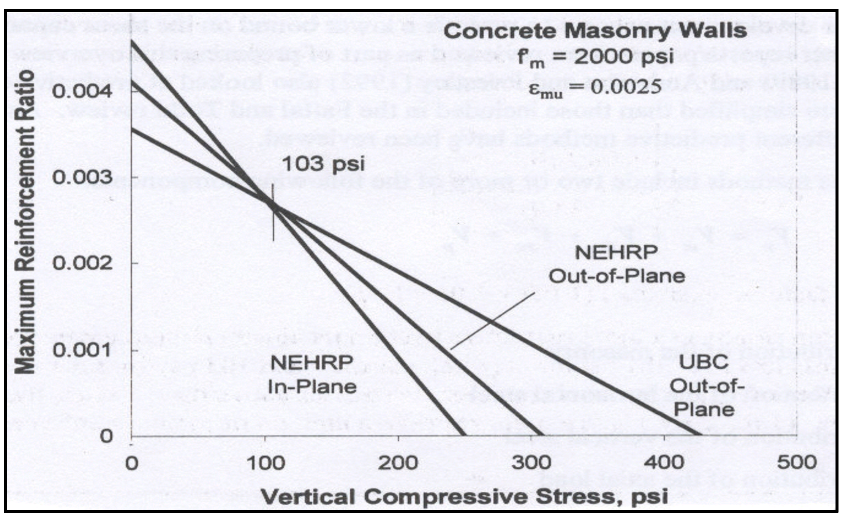

FIGURE C11.6.2.2-2 Maximum reinforcement for concrete masonry walls.

For further discussion, see He and Priestley (1992), Leiva and Klingner (1991), Limin and Priestley (1988), Merryman et al. (1989), Seible et al. (1992), and Shing et al. (1991).

11.6.3 Design of Plan (Unreinforced) Masonry Members:

11.6.3.5: The axial load strengths given by Eq. 11.6.3.5-1 and 11.6.3.5-2 are based on analysis of the results of axial load tests performed on clay and concrete masonry elements. For members having an h/r ratio not exceeding 99, the specimens failed at loads less than the Euler buckling load.

Eq. 11.6.3.5-1 was empirically fit to test data for these members. For h/r values in excess of 99, the limited test data is adequately approximated by the Euler buckling equation.

11.7 SHEAR:

11.7.3 Design of Reinforced Masonry Members: The development of strength design procedures for masonry requires a reasonably simplified and accurate equation that is capable of predicting the ultimate shear strength of a masonry wall. Once agreed upon, this equation, together with appropriate ϕ factors, will form a key part of strength design procedures.

Over the past two decades many hundreds of tests have been performed in the U.S., Japan and New Zealand to determine the strength and ductility of concrete block and clay brick shear walls subjected to cyclic lateral load patterns. From these tests come equations to predict the shear strength of walls usually calibrated to the tests carried out by the particular researcher. Fattal and Todd (1991) compared the predictions of four different equations with available experimental results. The only flaw in this work was that they included the UBC design equations with the inference that the UBC equations were predictive equations for the ultimate shear strength of masonry. This is not the intent of the UBC equations. They were developed and then modified as part of the code development process to provide a lower bound on the shear capacity of masonry walls. Two other reports/papers were reviewed as part of preparing this overview document; Blondet et al. (1989) and Anderson and Priestley (1992) also looked at predictive equations which were more simplified than those included in the Fattal and Todd review. As a consequence, a total of six different predictive methods have been reviewed.

In summary, the methods include two or more of the following components:

$$V_u = V_m + V_{sh} + V_{sv} + V_p \tag{C11.7.3-1}$$

where:

V_m = contribution of the masonry

V_{sh} = contribution of the horizontal steel

V_{sv} = contribution of the vertical steel

V_p = contribution of the axial load

The report by Fattal and Todd (1991) is quite thorough and the test data used to assess the Shing, Matsamura, and Architectural Institute of Japan (AIJ) predictive equations were also used to assess the methods proposed by Blondet et al. (1989) and Anderson and Priestley (1992) and the final TCCMaR equations that were developed as part of the TCCMaR study. The form of these equations are given in Table C11.7.3-2. Rather than present the details of each of the test results that were developed, a statistical summary is provided in Table C11.7.3-1. This provides the overall average, standard deviation and coefficient of variation for all 62 tests included in the Fattal and Todd report. The values given in Table C11.7.3-1 are the ratio of the shear strength obtained by the predictive equation divided by the ultimate strength obtained from the test. A perfect prediction has a ratio of 1 and a conservative prediction has a ratio less than 1.

TABLE C11.7.3-1

Tests	Shing	Okamoto	Matsamura	Blondet et al.	Anderson & Priestley	TCCMaR
All 62 tests						
Mean	0.83	0.81	0.91	1.03	1.06	1.02
Standard Deviation	0.23	0.27	0.20	0.24	0.23	0.24
Coefficient of Variation	0.05	0.07	0.04	0.06	0.05	0.05
Mean Values						
Tests 1-10 (Shing)	0.94	1.25	0.93	0.88	1.02	0.87
Tests 11-27 (Matsu-mura)	0.89	0.82	0.99	1.10	1.13	1.07
Tests 27-38 (Okamoto)	0.65	0.76	0.75	0.80	0.86	0.81
Tests 39-62 (Sveinsson)	0.82	0.66	0.91	1.13	1.11	1.12

Also included in Table C11.7.3-1 are the mean values of the four different sets of tests. Test 1-10 are from Shing et al. (1991), Tests 11-28 are from Matsamura (1987), Tests 29-37 are from Okamoto et al. (1987), and Tests 38-62 are from Sveinsson et al. (1985).

TABLE C11-7.3-2

	Masonry Component	Horizontal Steel	Vertical Load
TCCMAR	$\left(\frac{4-1.75M}{V_d}\right)\sqrt{f'_m}$	$0.5\rho_h f_h$	$0.25\sigma_c$
Blondet et al.	$\left(\frac{4-1.75M}{V_d}\right)\sqrt{f'_m}$	$0.5\rho_h f_h$	$\sqrt{V_m^2+\frac{V_m\sigma_c}{1.5}}-V_m$
Anderson	$2.9\sqrt{f_m'}$	$0.5\rho_h f_h$	$0.25\sigma_c$
Shing[a]	$(0.0217\rho_v f_v+0.166)\sqrt{f_m'}$	$\left(L-\frac{2d^1}{S_h}-1\right)\frac{S_h}{L}\rho_h f_h$	$(0.0217\sigma_c)\sqrt{f'_m}$

TCCMAR	$\left(\frac{4-1.75M}{V_d}\right)\sqrt{f'_m}$	$0.5\rho_h f_h$	$0.25\sigma_c$
Matsamura[a]	$\left[\left(\frac{0.76}{r_d+0.7}+0.012\right)\left(4.04\rho_v^{0.3}\right)\sqrt{f'_m}\right]\frac{d}{L}$	$\left[0.01575\left(\rho_h f_h\right)^{1/2}\sqrt{f'_m}\frac{\delta d}{L}\right]$	$\left(0.175\sigma_c\right)\frac{d}{L}$
AIJ	$\left[4.64\rho_v^{0.23}\left(0.01f'_m+0.176\right)\left(\frac{1}{R_c+0.12}\right)\right]\frac{d}{L}$	$\left[0.739\left(\rho_N f_h\right)^{1/2}+0.739\left(\rho_v f_v\right)^{1/2}\right]\frac{d}{L}$	$\left(0.0875\sigma_c\right)\frac{d}{L}$

[a]These equations are in metric units.

As part of the TCCMaR studies, it was decided to use a combination of the Blondet et al. and Anderson and Priestley equations. In comparing the manner in which the two methods account for contribution of the masonry component, it was decided to use the Blondet form. As part of the Berkeley tests (Mayes et al., 1976, Chen et al., 1978, Hidalgo et al., (1978, 1979), it was concluded that the M/Vd ratio should be part of the masonry equation rather than just a straight function of 2.9 $\sqrt{f'_m}$ as in the Anderson and Priestley equation. Furthermore, there was very little numerical difference in the values used to account for the vertical load contribution. As a consequence, it was decided to use the more simplified form of $0.25\sigma_c$ used by Anderson and Priestley. The final form of the TCCMaR equation was given as:

$$v = (4 - 1.75M/Vd)\sqrt{f'_m} + 0.5\rho_h f_{yh} + 0.25\sigma_c \qquad \text{(C11.7.3-2)}$$

The metric equivalent of Eq. C11.7.3-2 is:

$$v = 0.083(4 - 1.75M/Vd)\sqrt{f'_m} + 0.5\rho_h f_{yh} + 0.25\sigma_c$$

Some members of TCCMaR believed that some contribution of vertical steel should be included and this issue was investigated. Many of the test specimens only included jamb steel and consequently two different vertical steel contributions were investigated: $1/4\rho_v f_{yv}$ and $1/4\rho_{vi} f_{yvi}$ where ρ_v is the total vertical steel and ρ_{vi} is only the interior vertical steel and neglects the jamb steel. The correlation and the test results were not as good when a contribution from vertical steel was included and consequently it was decided not to include it in the recommended TCCMaR shear equation.

Application of the shear strength equation to partially grouted masonry was based in part on Fattal (1993a and 1993b).

11.8 SPECIAL REQUIREMENTS FOR BEAMS:

11.8.1: Masonry beams may be loaded normal to their plane by wind or earthquake forces. The beam must have adequate strength to span between support points under the action of the out-of-plane loads. The arbitrary limits of 50 and 32 were judged to be adequate absolute limits on the unbraced span to beam width ratios for the conditions listed.

11.8.2: Gravity loading of a masonry beam may be applied eccentrically to its vertical centroidal plane. The lateral supports of the masonry building should restrain the beam from rotation under the eccentric action of the gravity load.

If the beam is supported laterally at one edge only (top or bottom), then the lateral support should have the moment capacity to restrain the rotation caused by loading normal to the face of the beam that is eccentric to the support point.

11.8.3: A minimum amount of flexural reinforcement in the positive moment zone of the beam is specified. This minimum is specified as a ratio, ρ, of the quantity of the reinforcement to the cross-sectional area of the beam. The minimum ratio specified is intended to require that the post-cracked moment capacity exceeds the uncracked moment capacity of the section.

These requirements for a minimum quantity of positive moment reinforcement assumes that cracking has occurred in zones of negative moment and that the change in beam stiffness has increased the positive moment. However, if the positive moment capacity of the reinforced section exceeds the uncracked positive moment capacity, transfer of moment to this zone is accommodated.

If a section of the adjacent concrete floor serves as the compression flange of the beam, minimum reinforcement is based on the masonry section which is in tension due to positive moment.

11.8.4 Deep Flexural Members: The theory used for design of beams has a limited applicability to deep beams. Shear warping of the cross section and a combination of diagonal tension stress and flexural tension stress in the body of the deep beam requires that deep beam theory be used for design of members that exceed the specified limits of span to depth ratio. Analysis of wall sections that are used as beams generally will result in a distribution of tensile stress that requires the lower one-half of the beam section to have uniformly distributed reinforcement. The uniform distribution of reinforcement resists tensile stress caused by shear as well as flexural moment.

The flexural reinforcement for deep beams must meet or exceed the minimum flexural reinforcement ratio of Sec. 11.8.3. Additionally, horizontal and vertical reinforcement must be distributed throughout the length and depth of deep beams and must provide reinforcement ratios of at least 0.001. Distributed flexural reinforcement may be included in the calculations of the minimum distributed reinforcement ratios.

Flexural reinforcement that is lumped entirely at the bottom and/or top of a deep flexural member, however, should be ignored when calculating the distributed horizontal reinforcement ratio. In such a case, the lumped flexural steel must provide a minimum flexural reinforcement ratio of $120/f_y$ in accordance with Sec. 11.8.3. For Grade 60 steel, this requirement is equivalent to a minimum flexural reinforcement ratio of 0.002.

Although this flexural reinforcement ratio results in twice the ratio required by Sec. 11.8.4.3, the flexural steel is lumped at the top and/or bottom of the beam and is not uniformly distributed. Since the intent of Sec. 11.8.4.3 is to ensure a minimum quantity of uniformly distributed reinforcement throughout the depth of the deep beam, the lumped flexural steel is not considered when calculating the minimum distributed reinforcement ratios.

11.9 SPECIAL REQUIREMENTS FOR COLUMNS:

11.9.1: Maximum and minimum limitations on the area of longitudinal reinforcement for columns are traditional values that have been in codes for many years. Minimum areas are limited so that creep of the masonry, which tends to transfer load from masonry to reinforcing steel will not result in increasing the stress in the steel to yield level. The maximum area limitation represents a practical limit on the amount of reinforcing steel in terms of economy and steel placement. No testing or research has been done to justify changes in these traditional values.

11.9.2: The minimum number of bars in columns also is a traditional number. It is obviously appropriate, however, to suit rectangular or square column shapes and tying requirements.

11.9.3: The lateral tie restrictions in this section are also traditional. The column tie bending requirements of Part c are to be as shown.

Reinforcement is restricted to an amount below the area required for flexural bending only in order to preserve a ductile failure condition (i.e., steel will reach ultimate yield strain before concrete reaches ultimate yield strain which would be defined as a brittle failure). It is therefore important to keep the reinforcement ratio low.

11.10 SPECIAL REQUIREMENTS FOR SHEAR WALLS:

11.10.1 through 11.10.5: Detailing requirements for masonry shear walls have been reorganized for 1997 in Sec. 11.10.1 through 11.10.5 to provide direct correlations with those categories given as line items in Table 5.2.2: ordinary plain masonry shear walls, detailed plain masonry shear walls, ordinary reinforced masonry shear walls, intermediate reinforced masonry shear walls, and special reinforced masonry shear walls. This was done so that variable R, Ω, and C_d factors could be given for each shear wall category rather than specifying detailing requirements per the Seismic Design Category as was done in previous editions of the Provisions. This reorganization is more consistent with the other material chapters, which are organized by type of lateral-force-resisting elements (e.g., ordinary, intermediate, or special moment resisting frames).

The word “plain” refers to the condition when a wall is unreinforced or tensile stresses in reinforcement, if any, are neglected. The word “reinforced” refers to the condition when tensile stresses in reinforcement are considered in the design process. “Detailed plain” and “intermediate reinforced” walls much have minimum reinforcement per Seismic Design Category C whereas “ordinary plain” and “ordinary reinforced” walls do not need to have any minimum reinforcement. Reinforcement requirements for “special reinforced” walls follow the requ9irements for Seismic Design Categories D and E. Requirements in each Seismic Design Category that are not germane to masonry walls have bene retained in Sec. 11.3.5 through 11.3.9. in newly.

11.10.6 Flanged Shear Walls: Tests on flanged shear walls (Priestley and Limin, 1990; Sieble et al., 1992) have indicated that if the conditions of Sec. 11.10.3.1 are satisfied, the flange will act in conjunction with the web as a part of the flexural member.

The tributary flange widths defined in Sec. 11.10.3.3 and 11.10.3.4 are considered to be values appropriate for predicting flexural behavior and strength. The values were taken from experimental results. This has significance when calculating probable shear force on the wall, which is related to the probable maximum flexural strength. For the calculation of maximum allowable reinforcement ratios, the reinforcement in the flange of the width specified in Sec. 11.10.3.4 must be considered as part of the maximum reinforcement ratio.

11.10.7 Coupled Shear Walls: Coupled shear walls are defined as shear walls in a common wall plane that are interconnected or coupled by spandrel beams. These beams are typically at each floor level. The coupling beams can be a section of a reinforced concrete floor that has continuity with the shear walls. Caution should be exercised to distinguish between coupled shear walls and walls with openings. In a coupled wall system, the yield limit state is allowed only in the coupling beam and at the base of the shear wall. If the flexure or shear yield state occurs in the wall between coupling beams, the system is a wall with openings. This system has very limited ductility and should be redesigned to prevent yielding in the reinforced wall at points other than the base of the shear wall.

Conformance with the requirement that the coupling beams reach their moment limit state at or before the shear wall reaches its moment limit state need not be checked if the ratio of the depth of the shear wall to the depth of the coupling beams exceeds 3 or more and the length of the coupling beams is less than one-half of the story height. Linear elastic analyses of the coupled wall system are inadequate to determine the yield status of the shear wall and the coupling beams. The stiffness of the shear wall will degrade rapidly in the first story. The shear walls in the upper stories may be uncracked.

11.10.7.2 Shear Strength of Coupling Beams: The nominal shear strength of coupling beams must be equal to the shear caused by development of a full yield hinge at each end of the coupling beams. This nominal shear strength is estimated by dividing the sum of the calculated yield moment capacity of each end of the coupling beams, M_1 and M_2, by the clear span length, L.

A coupling beam may consist of a masonry beam and a part of the reinforced concrete floor system. Reinforcement in the floor system parallel to the coupling beam should be considered as a part of the coupling beam reinforcement. The limit of the minimum width of floor that should be used is six times the floor slab thickness. This quantity of reinforcement may exceed the limits of Sec. 11.6.2.2 but should be used for the computation of the normal shear strength.

11.12 GLASS-UNIT MASONRY AND MASONRY VENEER: Chapters 11 and 12 of ACI 530-95/ASCE 5-95/TMS 402-95 have been newly introduced in the 1997 Provisions to address design of glass-unit masonry and masonry veneer. Direct reference is made to these chapters for design requirements. Investigations of seismic performance have shown that architectural *components* meeting these requirements perform well (Jalil, Kelm and Klingner, 1992, and Klingner, 1994).

REFERENCES

Abboud, B. E., X. Lu, and F. C. Schmitt. 1993. "An Evaluation of Three Current Deflection Methods for Predicting the Lateral Deflection of Masonry walls." In Proceedings of the Sixth North American Masonry Conference, Philadelphia, Pennsylvania, pp.73-85.

Abboud, B. E., and A. A. Hamid. 1987. "A Comparative Study of the Flexural Behavior of Reinforced Block Masonry and Reinforced concrete Using Small Scale Model walls." In Proceedings of the Fourth North American Masonry Conference, Los Angeles, California.

Abboud, B. E., A. A. Hamid, and H. G. Harris. 1990. "Small Scale Modeling of Concrete Block Masonry structures," ACI Structural Journal 87(2):145-155.

Abboud, B. E. 1987. The Use of Small Scale Direct Models for Concrete Block Masonry Assemblages and Slander Reinforced walls Under Out-of-Plane Loads, Doctoral Thesis, Drexel University, Philadelphia, Pennsylvania.

Blondet, J. M., R. L. Mayes, T. E. Kelley, R. R. Villablanca, and R. E. Klingner. 1989. Performance of Engineered Masonry in the Chilean Earthquake of March 3, 1985: Implications for U.S. Design Practice. Austin: Phil Ferguson Structural Engineering Laboratory, University of Texas.

Brown, Russell H., and A. Rhett Whitlock. 1983. “Strength of Anchor Bolts in Grouted Masonry,” ASCE Journal of Structural Engineering 109(6) June.

CEB. 1995. “Design of Fastenings in Concrete (Draft CEB Guide, Parts 1 to 3)” and “Fastenings for Seismic Retrofitting (State-of-the-Art Report on Design and Application),” Task Group 3.5 (Embedments), Euro-International Concrete Committee (CEB), CEB Bulletin D’Information No. 226, Comite Euro-International du Beton, August.

Chen, S. W., P. A. Hidalgo, R. L. Mayes, R. W. Clough, and H. D. McNiven. 1978. Cyclic Loading Tests of Masonry Single Piers, Vol. 2, EERC Report UCB/EERC-78/27. Berkeley: University of California Earthquake Engineering Research Center.

Fattal, S. G. 1993a. strength of Partially Grouted Masonry shear walls, Report NIST 5147. Gaithersburg, Maryland: National Institute of Standards and Technology.

Fattal, S. G. 1993b. The Effect of Critical Parameters on the Behavior of Partially Grouted Masonry shear walls Under Lateral Loads, NIST Report NISTIR 5116. Gaithersburg, Maryland: National Institute of Standards and Technology.

Fattal, S. G., and D. R. Todd. 1991. Ultimate strength of Masonry walls: Prediction vs. Test Results, NIST Report NISTIR 4633. Gaithersburg, Maryland: National Institute of Standards and Technology.

Hamid, A. A., M. K. Hatem, H. G. Harris, and B. E. Abboud. 1990. Hysteretic Response and Ductility of Reinforced concrete Masonry walls Under Out-of-Plane Loading." In Proceedings of the Fifth North American Conference, Urbana, Illinois, pp. 397-410.

Hamid, A. A., B. E. Abboud, M. W. Farah, M. K. Hatem, and H. G. Harris. 1989. "Response of Reinforced Block Masonry walls to Out-of-Plane Static Loads," in U.S.-Japan Coordinated Program For Masonry Building Research Report 3.2(a), Drexel University.

He, L., and N. Priestley. 1992. Seismic Behavior of Flanged Masonry shear walls - Final Report, TCCMaR Report 4.1-2.

Hidalgo, P. A., R. L. Mayes, H. D. McNiven, and R. W. Clough. 1978. Cyclic Loading Tests of Masonry Single Piers, Vol. 1 and 3, EERC Report UCB/EERC-78/27 and 79/12. Berkeley, University of California Earthquake Engineering Research Center.

Horton, R. T., and M. K. Tadros. 1990. "Deflection of Reinforced Masonry Members," ACI Structural Journal 87(4):73-85.

Jail, I., W. Kelm, and R. E. Klingner. 1992. Performance of Masonry and Masonry Veneer Buildings in the Loma Prieta Earthquake. PMSEL Report 92-1. Austin: University of Texas, Department of Civil Engineering.

Klingner, R. E. 1994. Performance of Masonry Structures in the Northridge, California, Earthquake of January 17, 1991. Boulder, Colorado: The Masonry Society.

Leiva, G., and R. Klingner. 1991. In-Plane Seismic Resistance of Two-story Concrete Masonry shear walls with Openings, TCCMaR Report 3.1(c)-2.

Limin, H., and N. Priestley. 1988. Seismic Behavior of Flanged Masonry shear walls, TCCMaR Report 4.1-1.

Malik, J. B., J. A. Mendonca, and R. E. Klingner. 1982. “Effect of Reinforcing Details on the Shear Resistance of Short Anchor Bolts Under Reversed Cyclic Loading,” Journal of the American Concrete Institute, Proceedings, Vol. 79, No. 1, January-February, pp 3-11.

Matsumura, A. 1987. "Shear strength of Reinforced Hollow Unit Masonry walls." In Proceedings of the 4th North American Masonry Conference, Los Angeles, California.

Mayes, R. L., Y. Omote, and R. W. Clough. 1976. Cyclic Shear Tests of Masonry Piers, Vol. 1, Report 76-8. Berkeley: University of California Earthquake Engineering Research Center.

Merryman, K., G. Leiva, N. Antrobus, and R. Klingner. 1989. In-Plane Seismic Resistance of Two-story Concrete Masonry Coupled Shear Walls, TCCMaR Report 3.1(c)-1.

Okamoto, S., Y. Yamazaki, T. Kaminosono, M. Teshigawara, and H. Hirashi. 1987. "Seismic Capacity of Reinforced Masonry walls and Beams in Wind and Seismic Effects." In Proceedings of the 18th joint Meeting, U.S. - Japan Panel on Wind and Seismic Effects, ed. by N. J. Raufaste, Report NBSIR 87-3540. Gaithersburg, Maryland: National Institute of Standards and Technology.

Priestley, N., and H. Limin. 1990. "Seismic Response of T-Section Masonry shear walls." TMS Journal 9(1):10-19.

Seible, F., G. A. Hegemier, M. J. N. Priestley, G. R. Kingsley, A. Kurkchubasche, and A. Igarashi. 1992. The U.S. - TCCMaR Five-story Full Scale Masonry Research Building Test - Preliminary Report, TCCMaR Report 9.2-4.

Shaikh, A. Fattah. 1996. “Design of Hooked Anchor bolts in Concrete and Masonry: Proposed Code Provisions and Commentary,” prepared for the National Codes and Standards Council, 1996.

Shing, P. B., J. Noland, H. Spaeh, E. Klamerus, and M. Schuller. 1991. Response of Single-story reinforced masonry shear walls to In-Plane Lateral Loads, TCCMaR Report 3.1(a)-2.

Shing, P. B., M. Schuller, and V. S. Hoskere. 1990. "In-Plane Resistance of Masonry shear walls," ASCE Journal of Structural Engineering 116(3).

Sveinsson, B. I., H. D. McNiven, and H. Sucuoglu. 1985. Cyclic Shear Tests of Masonry Piers, Vol 4, Report UCB/EERC 85-15. Berkeley: University of California Earthquake Engineering Research Center.

Wolde-Tinsae, A. M., R. H. Atkinson, and A. A. Hamid. 1993. "State-of-the-Art Modulus of Elasticity of Masonry." In Proceedings of the Sixth North American Masonry Conference. Boulder, Colorado: The Masonry Society.

The following Technical Coordinating Committee for Masonry Research task reports not specifically cited providing the substantiating data for the strength design criteria presented in this chapter are available through the Earthquake Engineering Research Center Library in Richmond, California (phone 415-231-9403):

Task No. Author(s) and Title

1.1-1: Atkinson and Kingsley, Comparison of the Behavior of Clay & Concrete Masonry in Compression, September 1985. 151 pgs.

1.2(a)-1: Hamid, A. A., G. F. Assis, and H. G. Harris, Material Models for Grouted Block Masonry, August 1988. 67 pgs.

1.2(a)-2: Assis, G. F., A. A. Hamid, and H. G. Harris, Material Models for Grouted Block Masonry, August 1989. 134 pgs.

1.2(b)-1: Young, J. M., and R. H. Brown, Compressive Stress Distribution of Grouted Hollow Clay Masonry Under Strain Gradient, May 1988, 170 pgs.

1.3-1: Atkinson, R. H., An Assessment of Current Material Test Standards for Masonry Limit States Design Methods, June 1991. 38 pgs.

2.1-1: Hart, G., and M. Basharkhah, Slender Wall Structural Engineering Analysis Computer Program (Shwall, Version 1.01), September 1987. 68 pgs.

2.1-2: Hart, G., and M. Basharkhah, Shear Wall Structural Engineering Analysis Computer Program (Shwall, Version 1.01). September 1987. 75 pgs.

2.1-3: Nakaki, D., and G. Hart, Uplifting Response of Structures Subjected to Earthquake Motions, August 1987. 200 pgs.

2.1-4: Hart, G., N. Sajjad, and M. Basharkhah, Inelastic Column Analysis Computer Program (INCAP, Version 1.01), March 1988.

2.1-5: Hong, W. K., G. C. Hart, and R. E. Englekirk, Force-Deflection Evaluation and Models for University of Colorado Flexural walls, December 1989.

2.1-6: Hart, G. C., J. W. Jaw, and Y. K. Low, SCM Model for University of Colorado Flexural Walls, December 1989. 31 pgs.

2.1-7: Hart, G. C., N. Sajjad, and M. Basharkhah, Inelastic Masonry Flexural Shear Wall Analysis Computer Program, February 1990. 41 pgs.

2.1-8: Hart, G. C., R. Englekirk, M. Srinivasan, S. C. Huang, and D. J. Drag, Seismic Performance Study, DPC Gymnasium, Elastic Time History Analysis Using SAP90, February 1992. 41 pgs.

2.1-9: Hart, G. C., R. Englekirk, M. Srinivasan, S. C. Huang, and D. J. Drag, Seismic Performance Study, TMS Shopping Center, Elastic Time History Analysis Using SAP90, February 1992. 42 pgs.

2.1-10: Hart, G. C., R. Englekirk, J. W. Jaw, M. Srinivasan, S. C. Huang, and D. J. Drag, Seismic Performance Study, RCJ Hotel, February 1992. 51 pgs.

2.1-11: Hart, G. C., R. Englekirk, M. Srinivasan, S. C. Huang, and D. J. Drag, Performance Study, 2-Story Masonry Wall-Frame Building, February 1992. 112 pgs.

2.1-12: Hart, G. C., R. Englekirk, J. W. Jaw, M. Srinivasan, S. C. Huang, and J. D. Drag, Seismic Performance Study, Designed by Tentative Limit Sates Design Standard, February 1992. 75 pgs.

2.2-1: Ewing, R. D., A. El-Mustapha, and J. Kariotis, FEM/I - A Finite Element Computer Program for the Nonlinear Static Analysis of Reinforced Masonry Building Components, December 1987 (Revised June 1990). 124 pgs.

2.2-2: Ewing, R. D., Parametric Studies on Reinforced Masonry Shear Walls Using FEM/I, A Nonlinear Finite Element Analysis Program, March 1992.

2.2-3: Ewing, R. D., Finite Element Analysis of Reinforced Masonry Building Components Designed by a Tentative Masonry Limit States Design Standard, March 1992. 48 pgs.

2.3-1: Ewing, R., J. Kariotis, and A. El-Mustapha, LPM/I, A Computer Program for the Nonlinear, Dynamic Analysis of Lumped Parameter Models, August, 1987. 200 pgs.

2.3-2: Kariotis, J., A. El-Mustapha, and R. Ewing, Influence of Foundation Model on the Uplifting of Structures, July 1988. 50 pgs.

2.3-3: Kariotis, J., A. Rahman, and A. El-Mustapha, Investigation of Current Seismic Design Provisions for Reinforced Masonry Shear Walls, January 1990. 48 pgs.

2.3-4: Kariotis, J., A. Rahman, O. Waqfi, and R. Ewing, Version 1.03 LPM/I - A Computer Program for the Nonlinear, Dynamic Analysis of Lumped Parameter Models, February 1992. 227 pgs.

2.3-5: Kariotis, J., O. Waqfi, and R. Ewing, R., A Computer Program Using Beam Elements for the Nonlinear, Dynamic Analysis of Lumped Parameter Models, February 1992. 96 pgs.

2.3-6: Kariotis, J., and O. Waqfi, Comparison of the Dynamic Response of a Damped MDOF Nonlinear Beam Model with an Equivalent SDOF Hysteretic Model, April 1992. 88 pgs.

2.3-7: Kariotis, J., and O. Waqfi, Recommended Procedure for Calculation of the Balanced Reinforcement Ratio, February 1992. 73 pgs.

2.4(b)-1: Button, M. R., and R. L. Mayes, Out-of-Plane Seismic Response of Reinforced Masonry Walls: Correlation of Full-Scale Test and Analytical Model Results, March 1991. 65 pgs.

3.1(a)-1: Scrivener, J., Summary of Findings of Cyclic Tests on Masonry Piers, June 1986. 7 pgs.

3.1(a)-2: Shing, P. B., J. Noland, H. Spaeh, E. Klamerus, and M. Schuller, Response of Single-Story Reinforced Masonry Shear Walls to In-Plane Lateral Loads, January 1991. 136 pgs.

3.1(b)-1: Seible, F., and H. LaRovere, Summary of Pseudo Dynamic Testing, February 1987. 46 pgs.

3.1(b)-2: Igarashi, A., F. Seible, and G. Hegemier, Development of the Generated Sequential displacement Procedure and the Simulated Seismic Testing of the TCCMaR Three-Story In-Plane Walls, June 1993.

3.1(c)-1: Merryman, K., G. Leiva, B. Antrobus, and R. Klingner, In-Plane Seismic Resistance of Two-Story Concrete Masonry Coupled shear walls, September 1989. 176 pgs.

3.1(c)-2: Leiva, G., and R. Klingner, In-plane Seismic Resistance of Two-story Concrete Masonry Shear Walls with Openings, August 1991. 326 pgs.

3.2(a)-1: Hamid, A., B. Abboud, M. Farah, K. Hatem, and H. Harris, Response of Reinforced Block Masonry Walls to Out-of-Plane Static Loads; September 1989. 120 pgs.

3.2(b)-1: Agbabian, M., S. Adham, S. Masri, V. Avanessian, and V. Traina, Out-of-Plane Dynamic Testing of Concrete Masonry Walls, Vol. 1 and 2, July 1989. 220 pgs.

3.2(b)-2: Blondet, M., and R. L. Mayes, The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes, Vol. 1: General Information, April 1991. 172 pgs.

3.2(b)-2: Blondet, M., and R. L. Mayes, The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes, Vol. 2: Walls No. 4 and 6 (Group 1), April 1991. 267 pgs.

3.2(b)-2: Blondet, M., and R. L. Mayes, The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes, Vol. 3: Walls No. 8, 9, 10 and 11 (Group 2), April 1991. 310 pgs.

3.2(b)-2: Blondet, M., and R. L. Mayes, The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes, Vol. 4: Walls No. 3, 5, and 7 (Group 3), April 1991. 256 pgs.

4.1-1: He, L., and M. J. N. Priestley, Seismic Behavior of Flanged Masonry Shear Walls, May 1988. 119 pgs.

4.1-2: He, L., and M. J. N. Priestley, Seismic Behavior of Flanged Masonry Shear Walls - Final Report, November 1992. 279 pgs.

4.2-1: Hegemier, G., and H. Murakami, On the Behavior of Floor-to-Wall Intersections in Concrete Masonry Construction: Part I: Experimental.

4.2-2: Hegemier, G., and H. Murakami, On the Behavior of Floor-to-Wall Intersections in Concrete Masonry Construction: Part II: Theoretical.

5.1-1: Porter, M., and A. Sabri, Plank Diaphragm Characteristics, July 1990. 226 pgs.

5.2-1: Porter, M., F. Yeomans, and A. Johns, Assembly of Existing Diaphragm Data, July 1990. 142 pgs.

6.2-1: Scrivener, J., Bond of Reinforcement in Grouted Hollow-Unit Masonry: A State-of-the-Art, June 1986. 53 pgs.

6.2-2: Soric, Z., and L. Tulin, Bond Splices in Reinforced Masonry, August 1987. 296 pgs.

7.1-1: Paulson, T., and D. Abrams, Measured Inelastic Response of Reinforced Masonry Building structures to Earthquake Motions, October 1990. 294 pgs.

8.1-1: Hart, G., A Limit State Design Method for Reinforced Masonry, June 1988.

8.1-2: Hart, G., Expected Value Design in the Context of a Limit Sate Design Methodology, February 1990.

8.2-1: Hart, G., and G. T. Zorapapel, Reliability of Concrete Masonry Wall Structures, December 1991. 229 pgs.

8.2-2: Hart, G., and N. Sajjad, Confinement in Concrete Masonry, December 1990.

8.2-3: Hart, G., and J. Jang, Seismic Performance of Masonry Wall Frames, December 1991.

9.1-1: Kariotis, J. C., and A. W. Johnson, Design of Reinforced Masonry Research Building, September 1987. 42 pgs.

9.1-2: Kariotis, J. C., and O. M. Waqfi, Trial Designs Made in Accordance with Tentative Limit States Design Standards for Reinforced Masonry Buildings, February 1992. 184 pgs.

9.2-1: Seible, F., Report on Large Structures Testing Facilities in Japan, September 1985. 120 pgs.

9.2-2: Seible, F., Design and Construction of the Charles Lee Powell Structural Systems Laboratory, November 1986. 65 pgs.

9.2-3: Seible, F., The Japanese Five-story Full Scale Reinforced Masonry Building Test, January 1988. 100 pgs.

9.2-4: Seible, F., G. A. Hegemier, M. J. N. Priestley, G. R. Kingsley, A. Kurkchubasche, and A. Igarashi, The U.S. - TCCMaR Five-story Full Scale Masonry Research Building Test - Preliminary Report, October 1992. 58 pgs.

11.1-1: TCCMaR, Summary Report: U.S. Coordinated Program for Masonry Building Research, September 1985 to August 1986. 190 pgs.

11.1-2: TCCMaR, Status Report: U.S. Coordinated Program for Masonry Building Research, November 1988. 170 pgs.