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Seismic Design of Special Reinforced Masonry Shear Walls

A Guide for Practicing Engineers

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National Institute of
Standards and Technology
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NEHRP Seismic Design Technical Briefs

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A Guide for Practicing Engineers

Prepared for
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By
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Disclaimers

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NIST policy is to use the International System of Units (metric units) in all its publications. In this report, however, information is presented in U.S. Customary Units (e.g., inch and pound), because this is the preferred system of units in the U.S. earthquake engineering industry.

Cover photo. Construction of a full-scale, three-story special reinforced masonry wall structure for shake table tests at the Network for Earthquake Engineering Simulation site of the University of California at San Diego as part of a research project supported by the National Institute of Standards and Technology.

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1. Introduction

The primary seismic force-resisting elements in buildings are horizontal diaphragms, vertical framing elements, and foundations. Together, these elements, comprise the seismic force-resisting system (SFRS). In reinforced masonry structures, the vertical framing elements are generally structural walls. They resist out-of-plane loads from wind or earthquake and transfer those loads to diaphragms and foundations. They also resist in-plane loads received from diaphragms and convey them to foundations. Given the wide variety of masonry materials, forms, and local construction practices, many kinds of reinforced masonry structural walls are possible. This Guide focuses on the design of one classification of walls for one loading case: special reinforced masonry shear walls subjected to in-plane seismic and gravity loads. From the least to the most stringent ductility requirements, three subcatagories are defined: ordinary, intermediate, and special. Within the special classification, two fundamental types are distinguished:

- flexure-dominated walls: walls whose behavior is dominated by flexure, with reliable ductility and inelastic displacement capacity
- shear-dominated walls: walls whose behavior, often for reasons beyond the control of the structural designer, is dominated by shear, with limited ductility capacity.

Codes Referenced in this Guide

U.S. building codes are continually undergoing revisions to introduce improvements in design and construction practices. At the time of this writing, the building code editions commonly adopted by state and local jurisdictions include the 2012 edition of the *International Building Code* (IBC 2012), the 2010 edition of *Minimum Design Loads for Buildings and Other Structures*, ASCE 7 (ASCE 2010), and the 2011 edition of the Masonry Standards Joint Committee (MSJC) masonry code. To maximize the useful life of this Guide, it is written with reference to the latest MSJC masonry code, published in 2013 as *Building Code Requirements for Masonry Structures*, TMS 402-13/ACI 530-13/ASCE 5-13 (TMS 2013a). Collectively, these codes and standards are often referred to herein as "the code."

A note about the MSJC: the masonry code has traditionally been produced through the efforts of a joint committee sponsored by The Masonry Society (TMS), American Concrete Institute (ACI), and American Society of Civil Engineers (ASCE). In 2013, the ACI and the Structural Engineering Institute of the American Society of Civil Engineers (SEI/ASCE) released their rights to future editions of the building code and commentary TMS 402/ACI 530/ASCE 5 and the specifications TMS 602/ACI 530.1/ASCE 6 (TMS 2013b). Going forward, the masonry code will be known as "TMS 402" and the specifications as "TMS 602." This Guide respects that change.

This Guide uses "IBC" to refer to IBC 2012, "ASCE 7" to refer to ASCE 7 2010, "TMS 402" to refer to the 2013 TMS 402/ACI 530/ASCE 5, and "TMS 602" to refer to TMS 602/ACI 530.1/ASCE 6.

Other materials usually allow the structural designer to locate and size structural elements to achieve the desired or needed behavior, and the building is then constructed around these structural elements. Masonry, by contrast, serves simultaneously as architecture (defining a building's external or internal appearance as well as its internal functional program), enclosure (defining a building's external envelope), and structure (resisting vertical and lateral loads). The structural designer generally does not have the opportunity to choose the configuration of these wall elements; instead, the other design factors dictate their locations and proportions. Thus, the structural designer must work with the elements that configure the space. The designer must be able to anticipate the expected behavior of those elements so that he or she can adapt the design and detailing of each element appropriately to resist all required loading combinations to meet the intent of the code for stiffness, strength, and ductility. These requirements apply to structural walls in all Seismic Design Categories (SDC) as defined in ASCE 7, but can be particularly challenging for special walls because the expected level of ductility implied by the "special" designation may not be available.

Sidebars in this Guide

Sidebars are used in this Guide to provide additional guidance on good practices and open issues in analysis, design, and construction.

To keep this Guide to a manageable size, worked examples are not included. The reader is referred to the references for additional resources. Both The Masonry Designer's Guide, MDG-7 (TMS 2013c) and the text by Klingner (2010), although not completely current, address all aspects of masonry design according to TMS 402 in some detail with numerous examples. The general approach to capacity-based design adopted in this Guide is outlined in the text by Paulay and Priestley (1992).

Although special reinforced masonry walls can be used in any building, the IBC requires them only when masonry structural walls are used to resist seismic forces in new buildings assigned to SDC D, E, or F. The design force levels are specified in ASCE 7, and the design procedures and detailing requirements are addressed in the 2013 edition of TMS 402, *Building Code Requirements for Masonry Structures* (TMS 2013a). The masonry design requirements of these three codes or standards are generally consistent with respect to their design intent for flexure-dominated walls. In contrast, for shear-dominated walls, the assumed structural ductility associated with a particular response modification factor (*R*-factor) in ASCE 7 may not automatically result from the design and detailing requirements of TMS 402. This Guide provides guidance for both conditions.

This Guide is intended especially for the practicing structural engineer, although it will also be useful for building officials, educators, and students. Although it emphasizes code requirements and accepted approaches to their implementation, it also identifies good practices that go beyond the minimum requirements of the building code. Background information and illustrative sketches clarify the requirements and recommendations. Following the introduction, Sections 2 and 3 describe the use of structural walls in buildings and discuss intended behavior of these walls. Section 4 provides analysis guidance. Section 5 presents the design and detailing requirements of TMS 402 along with guidance on how to apply them. Section 6 presents additional requirements that must be considered for all masonry buildings, particularly those assigned to SDC D, E, or F. Section 7 addresses detailing and constructability challenges for special structural walls. **Figure 1-1** summarizes the design process described in detail in this Guide.

Items not addressed in this Guide

This Guide addresses hollow-unit concrete or hollow unit clay masonry, reinforced and grouted, as necessary to meet the requirements of special reinforced masonry structural walls. A number of masonry types are not addressed, including the following:

- Unreinforced masonry
- Prestressed masonry
- Masonry infill
- Hybrid masonry (masonry walls acting together with steel frames)
- Autoclaved aerated concrete (AAC) masonry
- Prefabricated masonry
- Masonry used as veneer or other nonstructural applications
- Empirically designed masonry
- Double-wythe, filled cavity brick masonry
- Confined masonry (walls constrained by reinforced concrete framing on all edges)

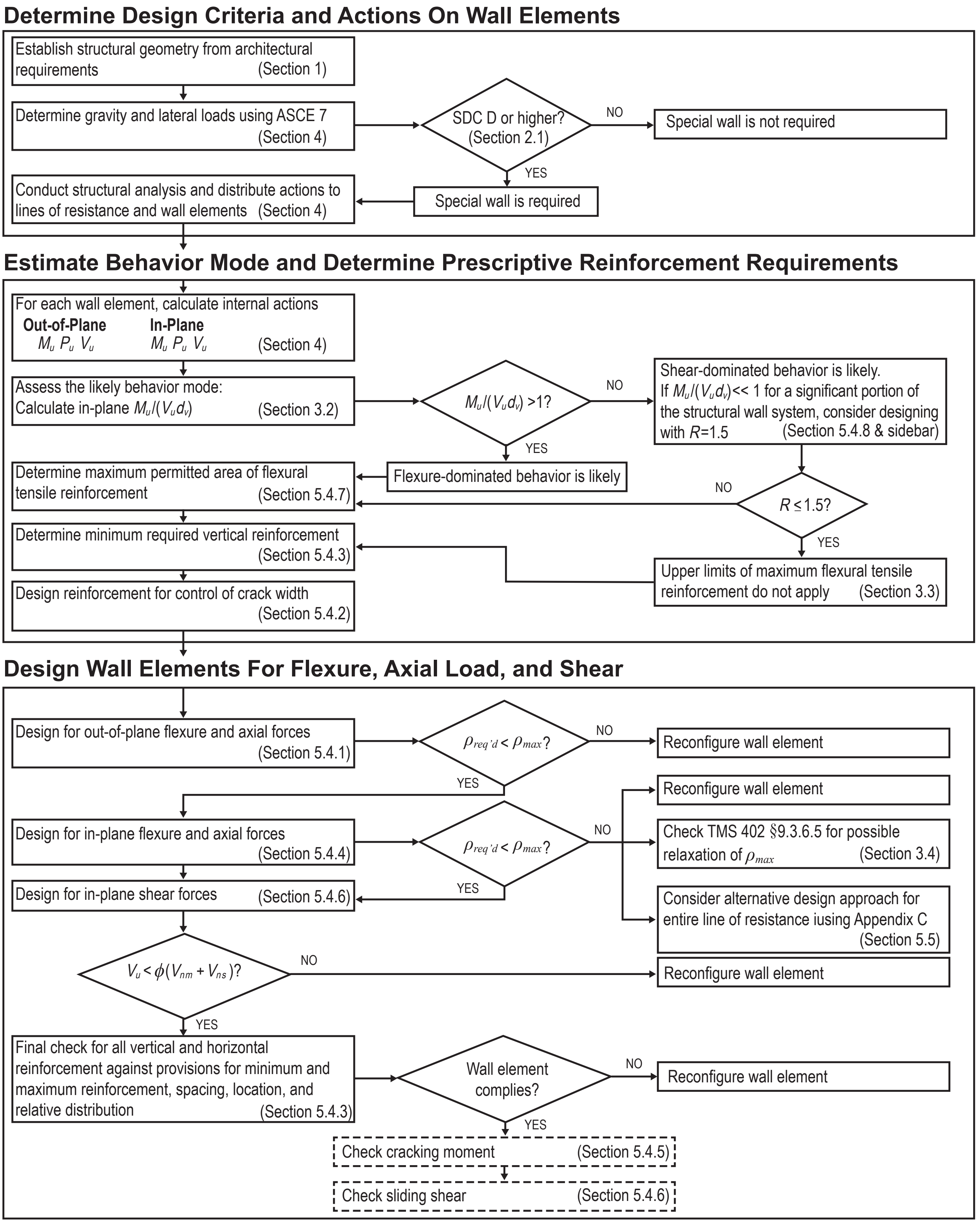


Figure 1-1. Flow chart of steps in the design of special reinforced masonry shear walls. Numbers in parenthesis cross-reference the sections in this Guide.

2. The Use of Reinforced Masonry Structural Walls in Buildings

This section focuses on the global behavior of groups of masonry walls acting together to form a vertical and lateral load-resisting system. It categorizes typical configurations of masonry structural walls in elevation and in plan and identifies structure and building types that are likely to have configurations that have significant structural consequences. Issues related to individual element design are deferred to Section 3.

2.1 Use of Special Reinforced Masonry Shear Walls

Masonry walls proportioned to resist a combination of shear, flexural, and axial forces are referred to as structural walls. When the primary function of structural walls is to resist in-plane loads conveyed from diaphragms down to the foundation, they are generally referred to as shear walls. ASCE 7 recognizes eight categories of masonry walls, and TMS 402 distinguishes among twelve shear wall categories. Within the category of reinforced masonry shear walls, three subcategories are recognized: ordinary, intermediate, and special. This Guide considers only special reinforced masonry shear walls.

Special reinforced masonry shear walls ("special walls") are required to meet the most restrictive material and prescriptive detailing requirements. Accordingly, they are permitted by ASCE 7 to be used in any SDC per the judgment of the structural designer. Special walls are required to be used for reinforced masonry walls in SDC D, E, or F.

Special walls are assigned the highest response modification factor, *R*, of any of the masonry shear wall types. For bearing wall systems, as defined by ASCE 7, special reinforced masonry shear walls are assigned an *R* factor of 5; for special reinforced masonry wall building frame systems, R = 5.5 (see sidebar next page.) Inherent in the use of an *R* factor of 5 or greater is the presumption of ductile behavior, associated with the development of plastic hinges with stable inelastic rotation capacity. Stable plastic hinges are characterized by the development of strains well past yield in the flexural reinforcement before the occurence of flexural strength degradation or shear failure occurs in the wall. It is the intent of prescriptive requirements found in TMS 402 to provide reinforcement configurations that ensure ductility. The prescriptive requirements of TMS 402 have been developed over many years. However, given the wide variety of masonry wall types and configurations and the lack of control of the structural designer over these configurations in many cases, the designer should not assume that following the prescriptive requirements alone will necessarily ensure ductile, flexure-dominated behavior.

2.2 Shear Wall Configurations in Buildings

A typical lateral load path through a masonry building is illustrated in **Figure 2-1**. Because masonry walls configure architectural space, they often perform as part of the building envelope or as fire or acoustic separation walls, in addition to their multiple structural roles. Walls often serve to resist both in-plane and out-of-plane forces from wind or seismic loads or both. In fact, once shear walls have been designed for out-of-plane forces, prescriptive reinforcement requirements, and shrinkage and thermal movements, walls often meet or exceed in-plane strength requirements.

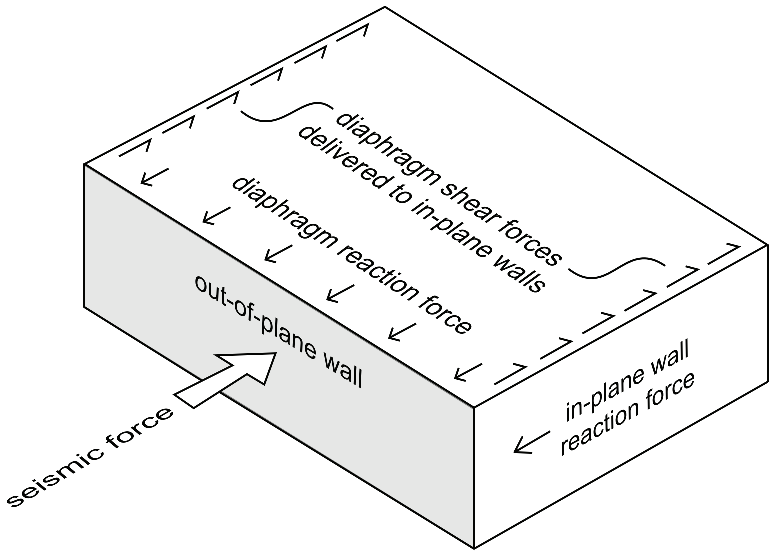


Figure 2-1. Typical load path through a masonry building.

Masonry walls can have a variety of plan configurations (**Figure 2-2**). Most reinforced masonry codes and design guides provide rules for the design of simple, planar wall elements as in **Figure 2-2(a)**, but in practice such walls can be part of complex structural elements and systems that affect their behavior, as illustrated in **Figures 2-2(b)** through **2-2(f)**. The designer can choose to design and detail such walls to have an integral cross section, with the wall segment aligned parallel to the lateral shear force acting as web and the perpendicular wall segments acting as tension or compression flanges as in **Figure 2-2(g)**. Alternatively, the designer can choose to treat groups of intersecting walls as individual planar elements, provided that they are sufficiently separated so that shear cannot be transferred between them either through the masonry or through stiff horizontal diaphragms. Depending on the nature of the diaphragm, small gaps between wall segments as in **Figure 2-2(h)** may not be sufficient to decouple the walls, and some separation may be required as in **Figure 2-2(i)**.

Typical wall configurations are shown in elevation in **Figure 2-3**. Squat wall elements like those in **Figures 2-3(a)** and **2-3(b)** with aspect ratios (height / plan length) of one or less are quite common, and they are often much stronger than required.

Building Frame versus Bearing Wall Systems

Masonry shear walls appear in two places in ASCE 7 Table 12.2-1, where building response modification factors are defined. They appear under “A. Bearing Wall Systems” and “B. Building Frame Systems.” The distinction between the two building systems, documented in the NEHRP 2003 Commentary (FEMA 2009), is that in Building Frame Systems the walls resist in-plane shear loads but only “a relatively small percentage” of gravity loads. This distinction may be interpreted differently by different jurisdictions. Generally, the intent of the original NEHRP definition of Building Frame Systems cannot be met unless the walls are specifically detailed to resist in-plane shear loads but not gravity loads. This Guide recommends the use of a Bearing Wall System, unless specific measures are taken to minimize the portion of gravity loads carried by walls relative to frames.

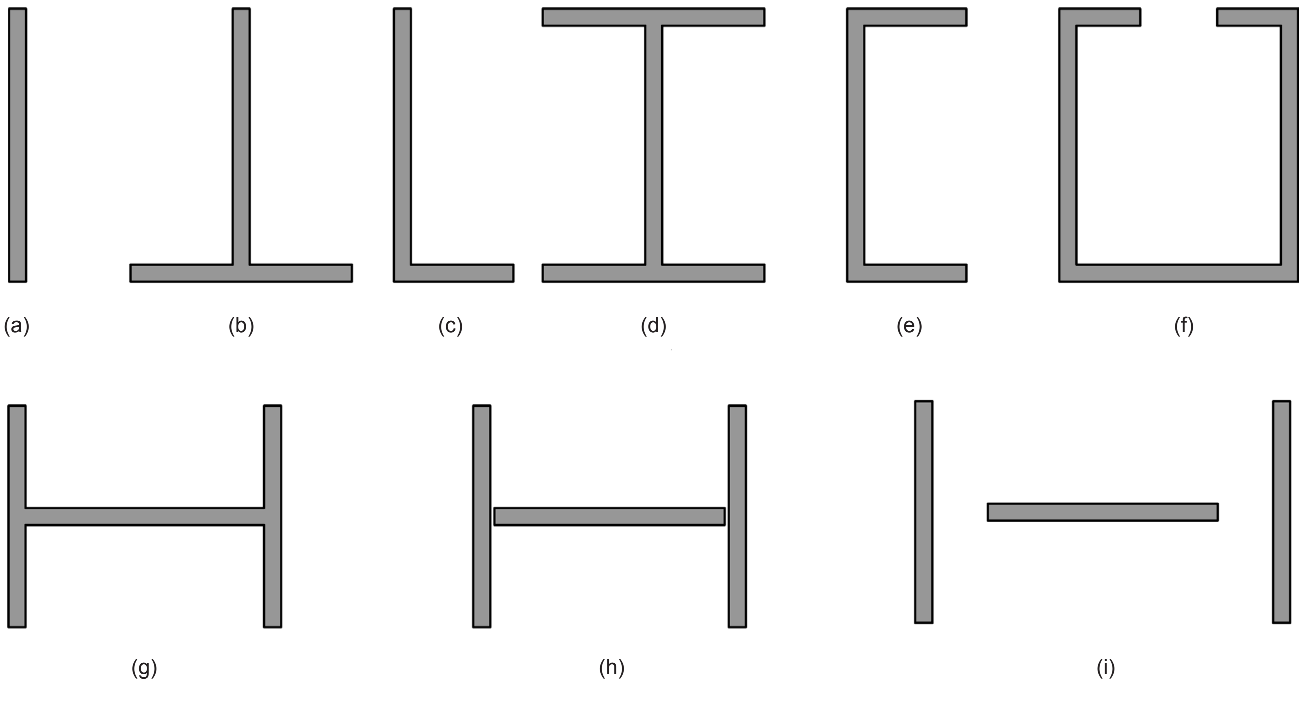


Figure 2-2. Plan configurations of walls: (a) typical shear wall, (b) T-shaped flanged wall, (c) L-shaped flanged wall, (d) I-shaped flanged wall, (e) C-shaped flanged wall, (f) box-section or core wall with an opening, (g) I-shaped wall with full continuity between web and flanges, (h) I-shaped wall with disconnected web and flanges and stiff coupling from floors, (i) I-shaped wall with flexible coupling from floors.

Another common wall type is the perforated wall, (**Figures 2-3(c)** and **2-3(d)**). These exhibit more complex behavior, depending on the governing behavior mode of individual pier and beam elements (vertically and horizontally oriented wall segments, respectively). Tall cantilever walls or cores (**Figure 2-3(e)**) are the configuration most likely to display the flexure-dominated behavior that meets the intent of the code for special walls. Multiple tall walls may be coupled by masonry beam elements (**Figure 2-3(g)**), which are introduced for architectural reasons such as fire separation. When the walls are subjected to significant lateral displacement, it is unlikely that these coupling beams can meet the demand for deformation without first failing in shear. In a more common configuration, walls are coupled only by concrete slabs (**Figure 2-3(f)**). Although the coupling effect of these slabs is often ignored in design, it can significantly increase the axial forces and moments generated in some walls. These issues are discussed in detail in Sections 3 and 4.

Common building types using the reinforced masonry special wall configurations in **Figure 2-3** are illustrated in **Figures 2-4** through **2-8**.

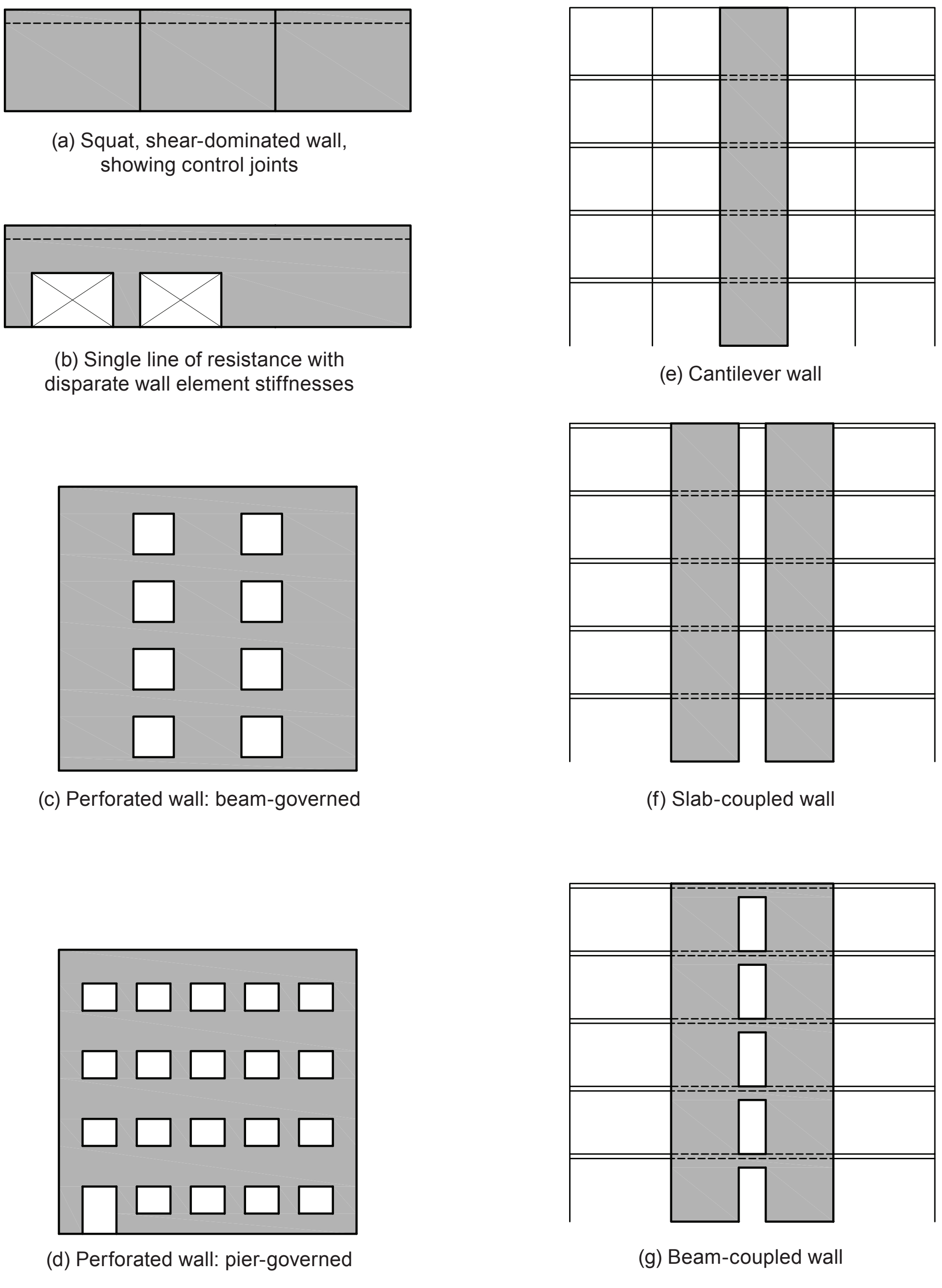


Figure 2-3. Elevations of typical masonry walls.

Figure 2-4. Squat, shear-dominated wall (typical of low-rise retail).

Figure 2-5. Line of resistance with disparate wall stiffnesses (typical of low-rise retail and industrial buildings).

Figure 2-6. Cantilever wall (residential with light wood framing).

Figure 2-7. Perforated wall (low-rise office or school).

Figure 2-8. Coupled wall with beam coupling (school).

Shear Walls and Structural Walls

The term "shear wall" has been commonly used for many years and is used in both TMS 402 and ASCE 7 to refer to walls that resist lateral seismic forces. Because the term is so common, it is also used in this Guide. In other sources, the reader may encounter the term "structural walls," which is used to make designers aware that "shear walls" may be dominated either by flexure or by shear.

3. Design Principles for Special Masonry Shear Walls

3.1 Allowable Stress Design, Strength Design, and Limit Design

TMS 402 offers both Allowable Stress Design (ASD) and Strength Design (SD) approaches. The ASD and SD approaches have been harmonized to produce similar designs for typical situations. The goal of TMS 402 is to allow the designer to use either method to achieve a design that is safe, constructible, and cost-effective. In this Guide, the emphasis is on SD because TMS 402 addresses ductility requirements relevant to special walls more explicitly for SD than ASD.

The 2013 edition of TMS 402 also includes a new Appendix C on Limit Design, which can be applied to individual lines of resistance in structures that are otherwise designed according to the SD requirements in Chapter 9. Limit Design allows the structural designer to explicitly take into account the anticipated plastic mechanism of the wall system, to control the aspect ratios and detailing of wall elements to achieve the best behavior possible, and to detail the elements in accordance with the resulting flexure- or shear-dominated behavior. Limit Design is particularly useful for perforated wall configurations that may have some shear-dominated wall elements (Lepage et al. 2011).

3.2 Flexure-Dominated versus Shear-Dominated Walls

A reinforced masonry wall system is composed of wall segments, each of which can be categorized as either flexure-dominated or shear-dominated. A flexure-dominated wall segment is one whose inelastic response is dominated by deformations resulting from the tensile yielding of flexural reinforcement. A shear-dominated segment is one whose inelastic response is dominated by diagonal shear (tension) cracks. A designer can check whether a wall segment is flexure-dominated or shear-dominated by comparing its flexural capacity with its shear capacity. A flexure-dominated segment has a lower flexural capacity than shear capacity; for a shear-dominated segment, the opposite is true.

Ductility is defined as the ability of an element to resist repeated reversed cycles of inelastic deformation without significant degradation of strength. The ductility capacity of flexure-dominated walls depends on the aspect ratio, the flexural reinforcement percentage, and the axial load. For example, **Figure 3-1** illustrates the theoretical relationship between ductility (in terms of curvature and in terms of displacement), flexural reinforcement percentage, and axial load for a wall segment with a shear-span-to-depth ratio $M_u/(V_u d_v)$ of 3.

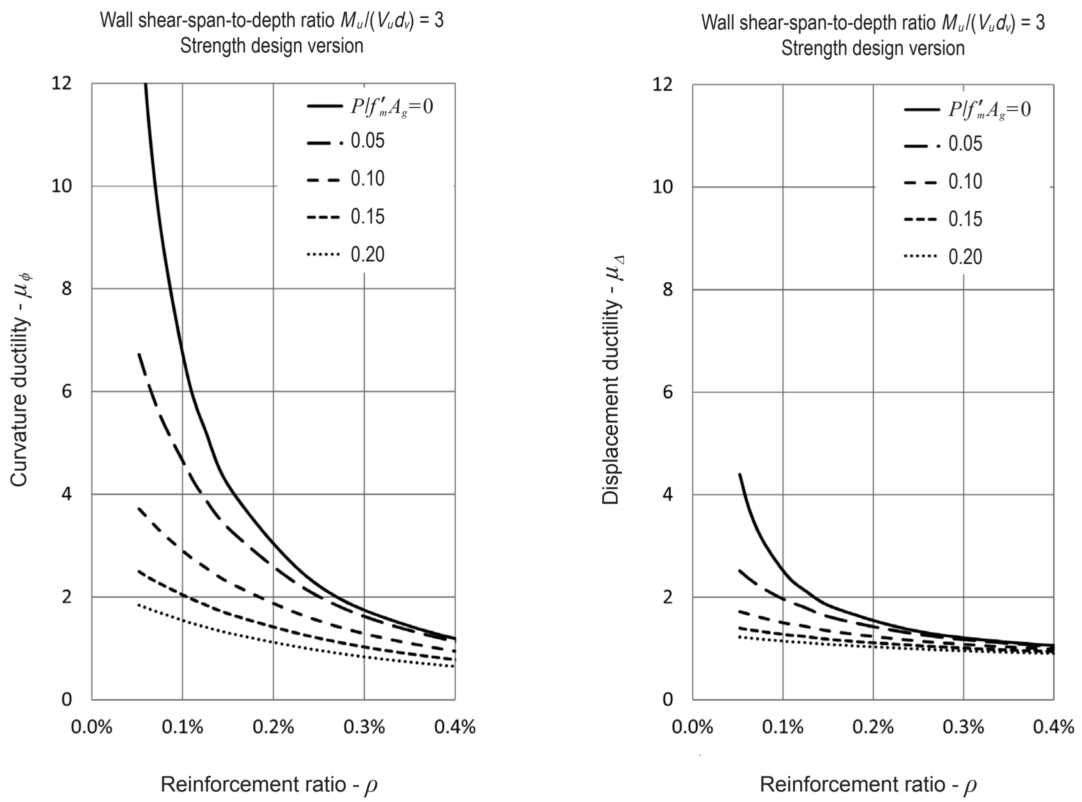
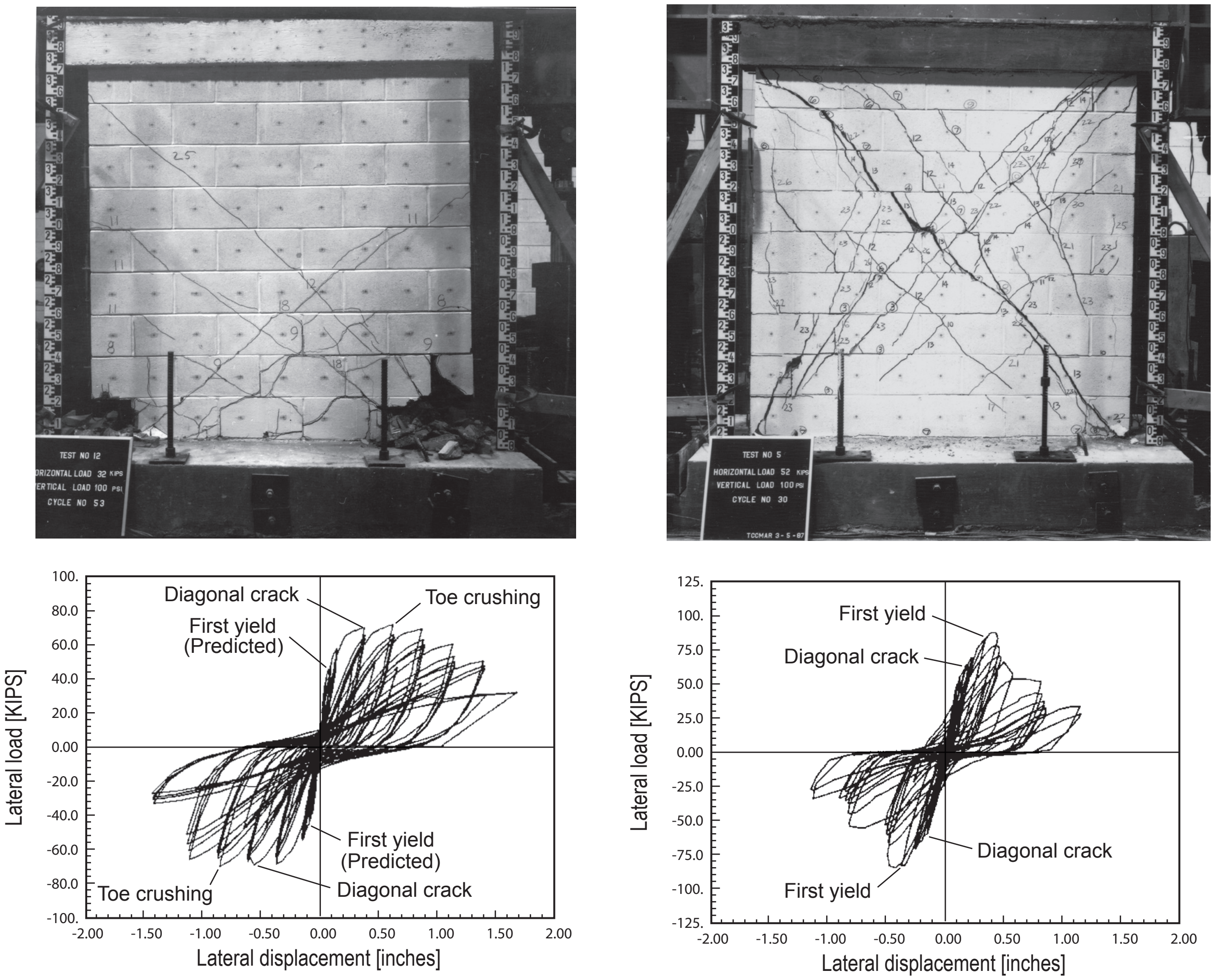


Figure 3-1. Influence of reinforcement ratio on curvature ductility and displacement ductility for varying levels of axial load on a wall with a shear-span-to-depth ratio $M_u/(V_u d_v)$ of 3.



(a) Flexure-dominated wall (b) Shear-dominated wall

Figure 3-2. Behavior of flexure-dominated and shear-dominated walls (Shing et al. 1989).

Flexure-dominated elements are generally ductile. Shear-dominated elements are generally brittle, with failure characterized by diagonal shear cracks. Flexure-dominated and shear-dominated behaviors are compared in **Figure 3-2**.

The implicit goal of TMS 402 is that special masonry shear walls be flexure-dominated and ductile. The code indirectly encourages designs that meet this goal through prescriptive requirements for distribution of reinforcement, limitations on bar diameters, maximum reinforcement restrictions, and other provisions, but these requirements may not be sufficient to produce ductile behavior. When a special shear wall has a shear-span-to-depth ratio $M_u/(V_u d_v)$ greater than one with a well-designed plastic hinge zone (Paulay and Priestley 1992), these requirements plus the required capacity-based design for shear generally result in flexure-dominated, ductile behavior. However, when a special shear wall has a shear-span-to-depth ratio less than one or a high axial load, the same combination of prescriptive requirements may still result in a wall that is shear-dominated and brittle. This is often the case for low-rise masonry buildings, which constitute most masonry construction in the United States.

Figure 3-3 illustrates how the behaviors of cantilever walls are influenced by the aspect ratio, axial load, and ratio of vertical to horizontal reinforcement. In this figure, $V_{flexure}$ is the shear demand associated with the expected flexural capacity, which is $1.25M_n$ divided by the wall height, with M_n being the nominal moment capacity, and V_{shear} is the nominal shear strength V_n calculated according to TMS 402. The figures are not design charts; they show the relative influence of those design parameters on the behavior mode. Data points above the line $V_{flexure}/V_{shear} = 1.0$ represent walls typically dominated by diagonal shear cracking. The behavior of squat walls is particularly sensitive to the change in axial load. **Figure 3-4** illustrates the same concepts with schematics, showing that shear-dominated behavior becomes more likely as the amount of vertical (longitudinal) reinforcement increases, the amount of transverse reinforcement decreases, the wall length increases, or the axial compression force increases.

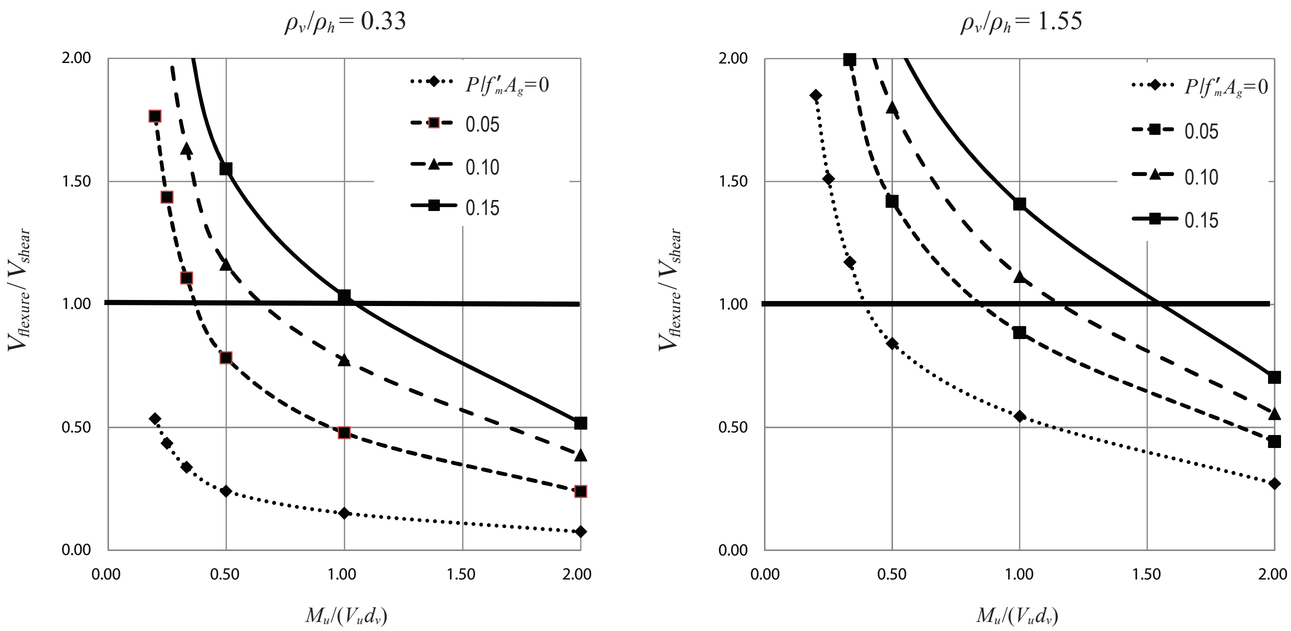


Figure 3-3. The effect of aspect ratio and axial load on the expected behavior of shear walls with two different ratios of vertical to horizontal reinforcement. Data points that fall above the line $V_{flexure}/V_{shear}$ = 1.0 represent walls most likely to have shear-dominated behavior.

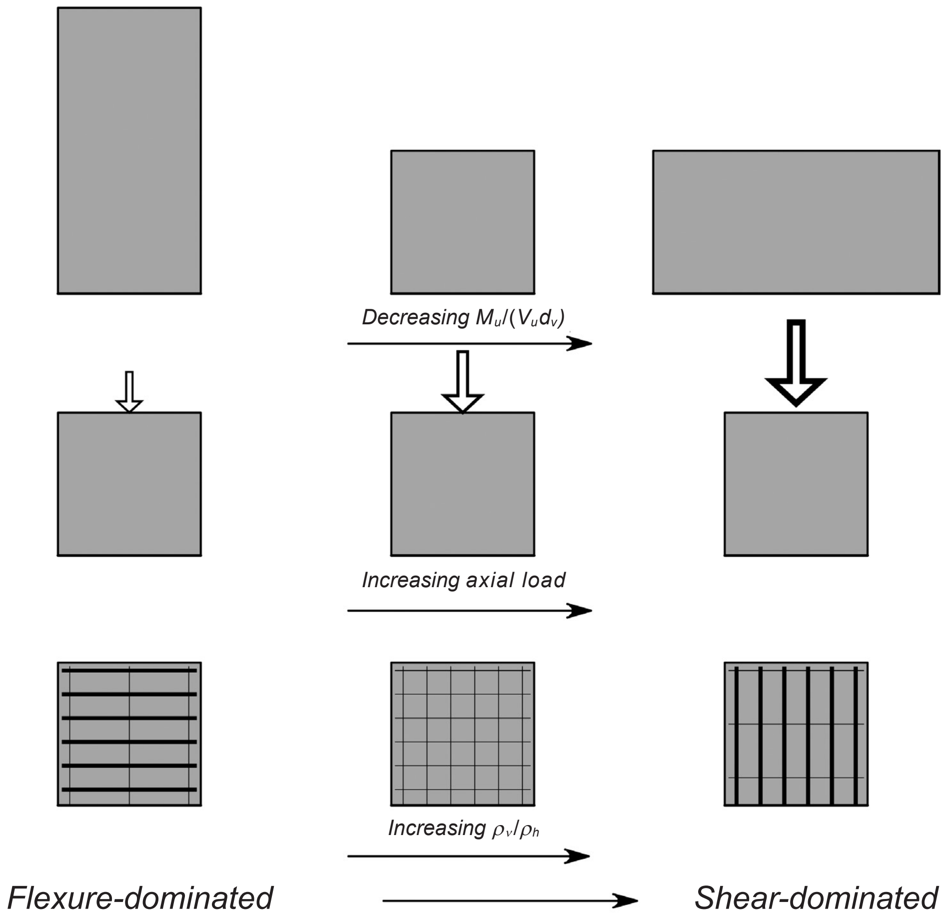


Figure 3-4. Conceptual illustration of the influence of shear-span-to-depth ratio $M_u/(V_u d_v)$, axial load, and ratio of vertical to horizontal reinforcement on wall behavior.

Figure 3-3 is based on a simple cantilever wall loaded at the top. In a real structure, numerous effects such as higher-mode effects or axial forces and moments induced by coupling elements can amplify the shear that can be developed, corresponding to the moment capacity of the wall beyond that represented here. Paulay and Priestley (1992) discuss various factors that can affect the failure mode in detail.

To protect a special wall against shear failure caused by possible flexural overstrength, TMS 402 §7.3.2.6.1.1 requires that the design shear strength, ϕV_n , exceed the shear corresponding to the development of the nominal moment capacity by a factor of at least 1.25. The code states that the nominal shear strength, V_n, need not exceed 2.5 times the factored shear demand V_u, but the designer should be aware that when the latter condition is invoked, shear-dominated behavior is likely.

When using ASD for special walls, TMS 402 §7.3.2.6.1.2 requires the shear or diagonal tensile stress resulting from in-plane seismic forces to be increased by a factor of 1.5 to encourage flexure-dominated behavior. This factor does not apply to the overturning moment.

3.3 Maximum Vertical Reinforcement Requirements

Because wall configurations are usually decided by the architect, and because selection of the masonry units, mortar, and grout are usually decided by the norms of local practice, the structural designer is left with the selection and detailing of reinforcement as the primary tool for producing a cost-effective masonry wall with the desired structural behavior. The code includes strict limits on maximum and minimum amounts of reinforcement.

The requirements of TMS 402 §9.3.3.5 for SD are intended to limit the amount of vertical reinforcement in shear walls to ensure that they exhibit ductile flexural behavior under seismic forces. The various limits of reinforcement stipulated in TMS 402 §9.3.3.5.1 through §9.3.3.5.4 are directly related to the respective ductility levels expected of ordinary, intermediate, and special walls. This is accomplished by specifying the minimum tensile strain that has to be developed in the extreme tensile reinforcement at the nominal moment capacity of the wall, and thereby to ensure that a minimum strain gradient can be attained by the wall section without severe crushing of the compression toe of the section. The minimum required tensile strain is a multiple, α, of the specified yield strain of the bar, $\varepsilon_y = f_y/E_s$. The factor α varies from 1.5 for ordinary walls (which are expected to have relatively low flexural ductility) to 3 for intermediate walls to 4 for special walls (which are expected to have relatively high flexural ductility). For walls loaded out of plane, α is 1.5 for all wall types.

Because of these requirements, as the design axial force increases, the maximum permissible reinforcement percentage decreases, as illustrated in **Figure 3-5**. Under some conditions, the maximum permissible reinforcement percentage can be zero or negative. In this case, the wall thickness must be increased (to decrease the axial stress), the specified compressive strength must be increased (to decrease P/f'_mA_g), or the wall configuration must be changed. **Figure 3-5** also shows that the maximum reinforcement percentage allowed for ordinary walls considerably exceeds that for special walls. However,

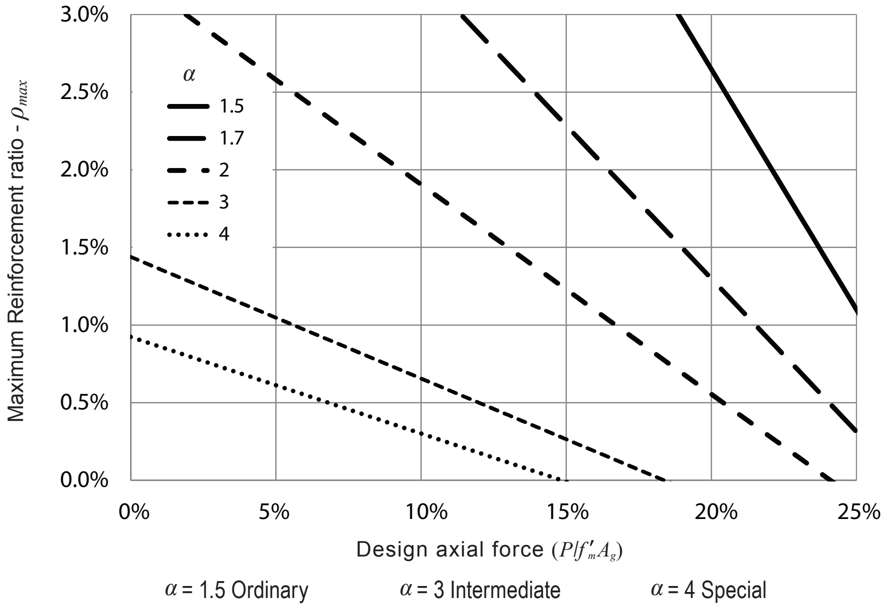


Figure 3-5. Maximum reinforcement ratios ρ_{max} using SD for in-plane walls with distributed reinforcement and varying levels of axial load, illustrating the effect of varying values for α. Only α values of 1.5, 3, and 4 are relevant to the code.

if a wall is also designed to resist out-of-plane loads, which is the usual case, the maximum reinforcement percentage for the out-of-plane condition can be the most limiting case. The equations in the commentary to TMS 402 §9.3.3.5 provide similar reinforcement limits for walls subjected to in-plane loading with distributed reinforcement and $\alpha = 4$ (which is the minimum required for special walls) and for walls subjected to out-of-plane loading, which has $\alpha = 1.5$. **Figure 3-6**, viewed together with **Figure 3-5**, shows combinations of bar size and spacing that meet the maximum reinforcement ratio for 8-inch walls with distributed reinforcement.

For squat walls with $M_u/(V_u d_v) < 1.0$, TMS 402 §9.3.3.5.4 allows the designer to design the wall for amplified forces–effectively, the forces associated with elastic response–in which case there is no upper limit to the maximum flexural tensile reinforcement. The maximum reinforcement provisions are also waived when the provisions for special boundary element reinforcement are satisfied (see the following section on special boundary elements).

Similarly, the ASD provisions in TMS 402 §8.3.4.4 have no maximum reinforcement limitations for shear walls with $M/(Vd_v) \leq 1.0$ and an axial load ratio $P/f'_m A_n \leq 0.05$. The ASD provisions have no maximum reinforcement limit for out-of-plane actions.

3.4 Special Boundary Elements

Boundary elements are frequently used in reinforced concrete structural walls to enhance flexural ductility by providing confinement in the compression zone and thereby allowing greater compressive strains. Although the potential benefits of boundary elements are equally clear for reinforced masonry walls, such boundary elements generally are not practical because of the restricted space inside a masonry wall. To achieve a sufficiently high volumetric ratio with sufficiently close spacing of transverse reinforcement within a typical masonry wall is challenging.

TMS 402 §9.3.6.5 has provisions for special boundary elements, but provides no guidelines for their design or detailing. When special boundary elements are used, TMS 402 requires that tests be conducted to verify that the strain capacity of the elements equal or exceed the compressive strain demand. These provisions were intended to serve as a gateway for future provisions addressing the behavior of boundary elements.

Studies have been conducted on the effectiveness of different confining schemes, including embedding steel plates (Priestley and Elder 1983) and open or closed wire mesh (Shing et al. 1993) in bed joints. These have been shown to enhance the

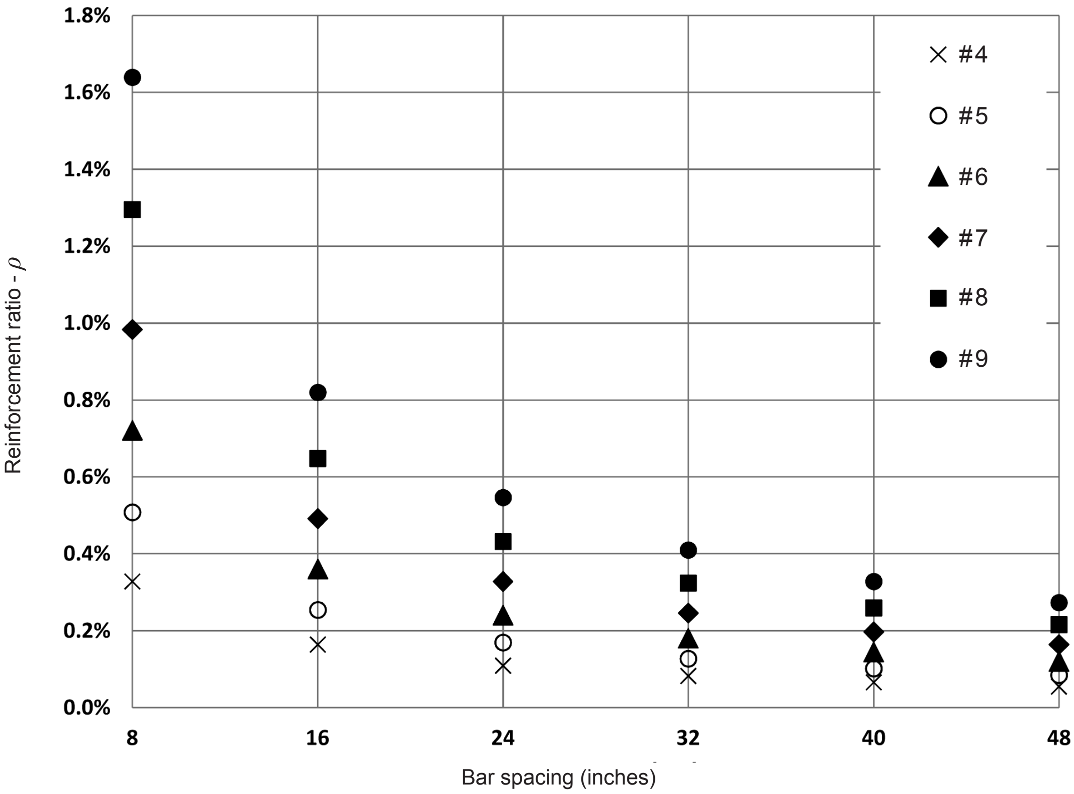


Figure 3-6. Reinforcement ratios for 8-inch masonry walls with uniformly distributed reinforcement and in-plane loads for different bar sizes and spacing with one centered bar per cell.

flexural ductility of reinforced masonry walls. However, steel plates are not convenient for construction, and the amount of confining steel that can be placed in bed joints is limited. The use of enlarged boundary elements constructed of hollow masonry units, similar to pilasters, has been shown to improve the flexural ductility of walls (Banting and El-Dakhakhni 2012), but such boundary elements may present architectural challenges and may be costly to build.

One way to resolve these issues in masonry walls is to use reinforced concrete boundary elements. This concept was studied by Cyrier (2012) as part of a recent research effort supported by NIST. The confinement details developed in that study are shown in **Figure 3-7**. Walls in this study were designed according to the requirements of ACI 318 §21.9.6.4 (ACI 2011) for special reinforced concrete walls. The boundary-element design with a return (**Figure 3-7(b)**) can also be used at the junction of intersecting walls. Boundary elements of this kind can significantly improve the flexural ductility of a wall.

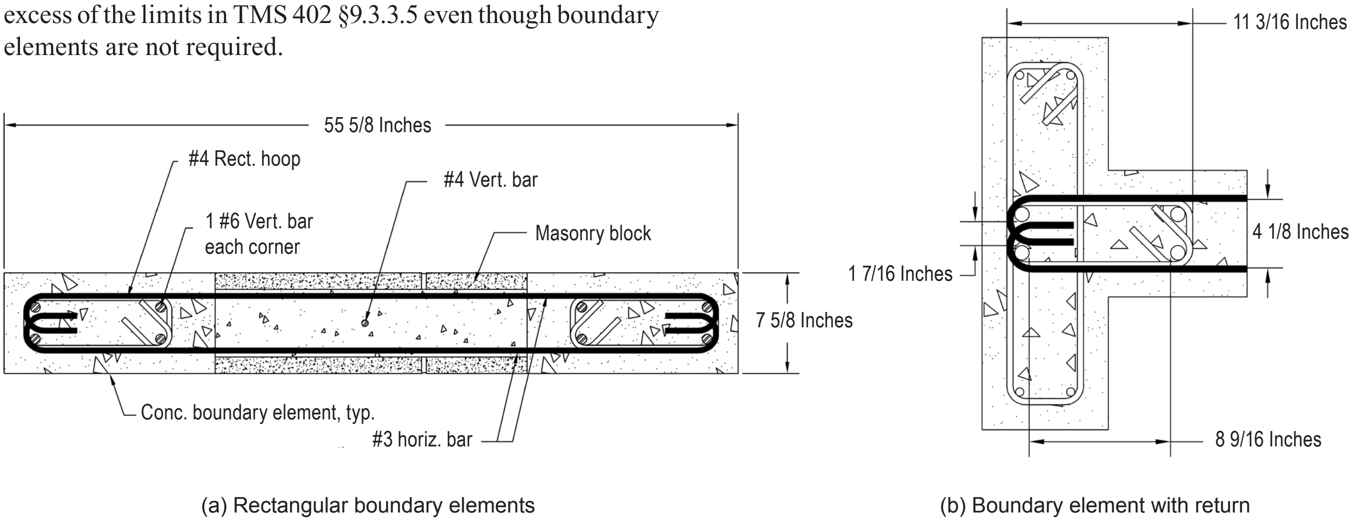
If the special boundary element provisions of TMS 402 §9.3.6.5 are satisfied, the code waives the maximum reinforcement requirements of TMS 402 §9.3.3.5. In TMS 402 §9.3.6.5.1, the code also establishes conditions under which these provisions are satisfied, without the need for special boundary elements. These include conditions in which a wall element is subjected to a low factored axial load, a wall element has a low to moderate shear-span-to-depth ratio ($M_u/(V_u d_v) \leq 1.0$), or a wall element is subject to low to moderate shear stresses. These conditions are met by many reinforced masonry structures. If none of these conditions is satisfied, the structural designer needs to perform one of two possible checks according to the code to see if special boundary elements are required. One is a displacement-based check the intent of which is to limit the ultimate curvature in the plastic-hinge region of the wall, and the other is a stress-based check. Satisfying these checks can sometimes allow the designer to specify reinforcement in excess of the limits in TMS 402 §9.3.3.5 even though boundary elements are not required.

3.5 Distribution of Reinforcement in Flexure-Dominated Walls

The normal and reasonable inclination of many structural designers is to concentrate vertical reinforcement in walls at the ends of the walls, where logically one would expect it to be most effective in increasing flexural capacity. For flexure-dominated special masonry shear walls with low axial loads, a number of factors suggest that a uniform distribution of smaller-diameter bars over the length of the wall is preferable to concentrating bars at the ends. Paulay and Priestley (1992) show that the moment capacities of two walls with the same total reinforcement, one having that reinforcement distributed uniformly and the other having that reinforcement concentrated at the wall ends, are nearly identical for typical levels of axial load and vertical reinforcement. Experimental results from the Technical Coordinating Committee for Masonry Research (TCCMAR) (Noland and Kingsley 1995) generally corroborate this conclusion. In general, uniform distribution of reinforcement encourages distributed, smaller flexural cracks and leads to less congestion of reinforcement, easier grout placement, and better construction quality. The shear-friction resistance at the base of a wall is also improved by distributed reinforcement, which provides better dowel action and a better clamping force mechanism, as discussed in Section 5 of this Guide. However, in some cases the maximum reinforcement limitations of the code may lead to vertical reinforcing bars that is concentrated at wall ends.

3.6 Grout Placement and Behavior

Grout, which surrounds the steel reinforcing bars and binds them to the masonry units, is critical to the behavior of special masonry walls. Fully grouted walls are those in which every cell, with or without reinforcement, is solidly filled with grout; partially grouted walls are those in which only the



(a) Rectangular boundary elements

(b) Boundary element with return

Figure 3-7. Reinforced concrete boundary elements in masonry walls (Cyrier 2012).

cells or locations with reinforcement are grouted, leaving sections of wall between lines of reinforced cells hollow. Partially grouted walls are generally considered to be more economical than fully grouted walls, and are the norm in the midwestern and eastern parts of the United States. In high seismic regions, fully grouted walls are more common. For heavily loaded, flexure-dominated special shear walls with significant ductility demand, fully grouted construction may be required. Conversely, partial grouting is appropriate for lightly loaded walls.

Effective and economical grout placement is largely a function of local practice and contractor preferences. Design choices affect grout placement technique: TMS 402 §3.2.1 controls the maximum height of grout pours based on the minimum dimensions of grout spaces and clearances around reinforcing bars. Some contractors choose to place grout in low lifts as the wall is constructed, using the minimum bar length possible from the lift below into the lift above; this can result in numerous lap splices and congestion within the wall. (See Section 3.7 on lap splices in plastic hinge zones.)

Recent Research on Partially Grouted Masonry

Tests conducted by Minaie et al. (2010) on partially grouted special walls showed that the shear strength equation provided in the 2011 and earlier editions of TMS 402 was unconservative for partially grouted walls. This problem has been addressed in the 2013 edition of TMS 402 by introducing a reduction factor of 0.75 to the shear strength calculated for partially grouted masonry. However, there are few data on the flexural behavior of partially grouted masonry. The ductility of partially grouted walls is expected to be less than that of fully grouted walls.

3.7 Lap Splices in Plastic Hinge Zones

Providing dowels at the foundation level that are lapped with the wall flexural reinforcement in the wall segment above, with similar laps at each floor level, is standard practice. The choice of grouting method may increase the number of laps within a wall segment, as discussed previously. These reinforcement lap splices are of little consequence in most reinforced masonry walls, but for flexure-dominated special walls, the designer may want to avoid locating lap splices in critical zones where flexural yielding is intended to occur.

The use of lap splices in plastic hinge zones is the subject of debate, and different design documents address such use differently. Although TMS 402 is silent on the issue, ASCE 7 §14.4 prohibits lap splices in plastic hinge zones. However, the 2012 IBC §1613.1 contradicts this requirement by explicitly excluding Chapter 14 of ASCE 7 and thus the prohibition of lap splices.

The negative effect of lap splices in flexure-dominated walls is based on several factors: flexural overstrength because of doubled reinforcement in the lap region; reduced available plastic rotation capacity because the presence of the splice reduces the length of the bar that can yield; the lack of transverse confinement of the splice compared to reinforced concrete walls; and potential slip of the splice. However, these factors may not cause a significant reduction in inelastic deformation capacity.

3.8 Wall Configurations and Behavior

Reinforced masonry shear walls can have different configurations as shown in **Figure 2-3**. For design purposes, they are classified into three types: cantilever walls, coupled walls, or perforated walls, based on their anticipated lateral force-resisting mechanisms. Except for the configurations in **Figures 2-3(a)** and **2-3(e)**, the distinction among them may not always be clear-cut. For example, a study by Seible et al. (1991) has shown that for concrete slabs consisting of hollow-core planks running parallel to in-line walls and covered by cast-in-place topping, if the concrete elements between the planks above door openings do not have transverse reinforcement, the slabs could experience brittle shear failure between the openings as the drift of the walls increases. In this case, the coupling effects of the slabs cannot be relied upon and the walls shown in **Figure 2-3(f)** should be treated as cantilever walls. However, the study has also shown that if the concrete elements have adequate transverse reinforcement, the slabs can behave as ductile coupling elements. For planks running perpendicular to the walls, coupling forces cannot be transmitted across the planks and should be ignored.

Most often, structural designers ignore the coupling effects of the slabs, and design walls with configurations such as that illustrated in **Figure 2-3(f)** as cantilever walls. However, this must be done with caution. In a recent study, a full-scale, three-story special reinforced masonry wall system constructed of concrete masonry units 8-inches thick, with concrete slabs consisting of hollow-core planks 6-inches thick running parallel to the in-plane walls and topping 3-inches thick, was tested on a shake table (Ahmadi 2012). The wall system had two door openings 40 inches wide in each story, and the reinforced masonry lintel above each opening was securely connected to the slab with dowels. The walls were designed as cantilever walls because control joints were introduced above the door openings and because the horizontal reinforcement in the lintels was debonded in the vicinity of the control joints. In spite of this, strong coupling actions were observed, eventually leading to the shear failure of two of the walls in the first story as shown in **Figure 3-8**. Nevertheless, the structure performed satisfactorily under the maximum considered earthquake because the reduced wall ductility was compensated for by a substantial increase in lateral resistance due to the coupling actions.

Figure 3-8. Shear failure of reinforced masonry walls in the first story during a shake table test.

A perforated wall is generally defined as a wall whose openings have dimensions that are small compared to the dimensions of the piers and beams around the openings (**Figures 2-3(b)** through **2-3(d)**). The behavior of these walls can be governed by flexure-dominated or shear-dominated behavior either in the piers or in the beams. Because of the low aspect ratios of the piers and beams, brittle shear behavior is highly possible in these elements. In fact, the behavior of the wall system shown in **Figure 3-8** resembles that of a perforated wall. Hence, it is sometimes difficult to make a clear distinction between a coupled wall and a perforated wall. For the purpose of the following discussion, a coupled wall is defined as a wall whose behavior is largely influenced by plastic hinging in ductile coupling elements. However, if the coupling elements are strong enough that plastic hinging or brittle shear behavior is expected in the vertical wall elements or piers, then it is considered to be a perforated wall. For cantilever walls, coupling effects are negligible.

While the structural designer must determine the most probable lateral load-resisting mechanism that governs the wall system as a first step in design, the SD approach has the advantage that it naturally allows and encourages the designer to check other probable mechanisms and detail the wall elements accordingly to prevent undesired consequences. This is desirable in view of various uncertainties in the behavior of reinforced masonry wall systems discussed above.

3.9 Design of Coupled Walls

According to the definition in the previous section, coupled masonry walls are characterized by ductile coupling elements, which can be concrete slabs alone or slabs plus reinforced masonry beams (lintels) as shown in **Figures 2-3(f)** and **2-3(g)**. These systems have a major advantage in that the coupling forces can reduce the moment demands on the walls and thereby result in a more economical design (Paulay and Priestley 1992). However, for tall flexure-dominated walls, the local ductility demand on the coupling elements can be very high, with local rotations being a multiple of the wall rotation, depending on the geometry. It is generally more difficult to reinforce masonry coupling beams to develop ductile, flexure-dominated behavior than it is for concrete coupling beams. For this reason, the coupling effects of these masonry coupling beams cannot be relied upon. On the other extreme, coupling beams can be so strong that they force flexural or shear yielding to occur in the walls. This can be the case when reinforced masonry beams are connected to the concrete slabs to form stiff and strong T- or L-shaped beams and the walls are relatively slender in comparison. When this happens, the wall system behaves like a perforated wall.

Walls connected by slabs alone are quite common. While flexural coupling by slabs is usually assumed to be relatively weak and often ignored in analysis, research (Paulay and Priestley 1992; Seible et al. 1991; Kingsley et al. 1994; Merryman et al. 1990) has shown that when multiple levels of weak coupling accumulate, they can contribute significantly to the overturning moment resistance of the wall system. This can be achieved with cast-in-place reinforced concrete slabs (Merryman et al. 1990) or hollow-core planks parallel to the walls with concrete topping, provided the concrete elements between the planks above door openings have adequate transverse reinforcement. In the 5-story building test (Kingsley et al. 1994) with topped hollow-core precast plank slab coupling, the coupling forces contributed more than half of the total overturning moment capacity of the system. Nevertheless, it must be cautioned that this can significantly increase the shear demand on individual walls.

Guidelines on estimating the effective width of coupling slabs and on the use of a limit analysis method to design coupled walls are presented in Section 4 of this Guide.

3.10 Design of Perforated Walls

Code provisions for special reinforced masonry wall design are intended to achieve a ductility level that can be realized only in flexure-dominated walls, and that is often unattainable in a perforated wall. In a perforated wall, wall elements between openings normally have low shear-span-to-depth ratios, and they are therefore vulnerable to brittle shear failure even though they satisfy the prescriptive reinforcing requirements and the shear capacity design requirements of TMS 402. When one or more vertical wall segments in the line of resistance has a design shear strength, ϕV_n, less than 1.25 times the factored shear demand corresponding to its nominal flexural capacity, shear-dominated behavior is possible even if the nominal shear capacity of each segment exceeds 2.5 times the factored shear demand. To avoid this situation, the shear resistance of the entire line of wall segments should be increased, resulting in a lower effective R factor. See **Figure 1-1**. If this is not done, the drift demand could exceed the local deformation capacities of shear-dominated wall segments, and their lateral shear capacities could diminish significantly during seismic response. As a result, most of the lateral seismic forces would shift to the flexure-dominated segments, which could lead to a story mechanism at that level. Story mechanisms of this kind are undesirable because they generally result in a concentration of drift demand and a consequent failure of all wall segments at that level. Alternatively, the design approach provided in Appendix C (Limit Design) of TMS 402 may be used, as discussed in the following section.

3.11 Limit Design Method

Appendix C (Limit Design) of TMS 402 provides an alternative way of designing special walls for the design seismic actions calculated using ASCE 7. Instead of distributing design story shears to wall segments according to the effective elastic stiffnesses of cracked walls, shears are permitted to be distributed according to plastic capacities. Although the plastic capacities are computed using the SD provisions of Chapter 9 of TMS 402 §9.3.3.5 (maximum permissible longitudinal reinforcement) and TMS 402 §9.3.6.5 (boundary elements) do not apply. Limit Design requires the structural designer to identify potentially controlling yield mechanisms in groups of wall segments under seismic actions, and it permits the designer to configure and reinforce those wall segments in a manner consistent with a preferred mechanism. The inelastic deformations of wall segments are prohibited from exceeding the maximum permitted values, which differ depending on whether the segments are flexure-dominated or shear-dominated. Limit Design accounts for brittle shear behavior in wall segments and limits the usable shear strength of shear-dominated wall segments to one-half that calculated according to TMS 402 §9.3.4.1.2. To determine the required design strengths of each wall segment, Limit Design requires plastic limit analysis, discussed in more detail in Section 4 of this Guide.

Although Limit Design is essentially a force-based method, it can be considered as an intermediate step toward displacement-based design in the way it controls inelastic deformations of individual masonry wall elements. Specific displacement-based design procedures have been proposed for reinforced masonry walls (Ahmadi et al. 2014a and 2014b), but have not been implemented in the United States.

3.12 Stiffness and Drift Limits

As part of its seismic design requirements, TMS 402 §7.4.3.2.4 requires that at any level or along any line of resistance at a particular story level, at least 80 percent of the lateral stiffness be provided by seismic force-resisting walls. The intention is to improve the accuracy and predictability of analysis and to ensure that elements, such as columns that are included for vertical capacity, do not form a significant part of the SFRS.

Story drift limits are established in TMS 402 §7.2.4, where they are tied to the governing building code requirements, or to the allowable story drift limits in ASCE 7. Story drift is limited (a) to control inelastic strain within affected elements, (b) to limit secondary moments because of P-Δ effects, and (c) implicitly, to control damage to both structural and nonstructural elements. Except in some cases for very tall, flexure-dominated walls, these story drift limitations are rarely a problem for masonry structures. The designer should also be aware of the importance of adequate separation between adjacent buildings. For that analysis, the total drift–as opposed to the story drift–is relevant.

3.13 Cracking Moment

Although TMS 402 §9.3.4.2.2.2 requires that the nominal capacity of a beam be not less than 1.3 times the cracking moment, TMS 402 does not impose this requirement for walls. This apparent inconsistency has been extensively discussed over the years. As of this writing, the prevailing opinion has been that such a requirement is not necessary for walls because the dynamic actions associated with flexural cracking of a wall under seismic excitation are transient and because they would not cause the brittle failure of an inadequately reinforced wall in the same way that gravity loads can cause the brittle failure of an inadequately reinforced beam. It has also been argued that walls typically have horizontal cracks at floor levels because of out-of-plane moments.

Nevertheless, some design offices recommend that walls have sufficient vertical reinforcement so that their nominal in-plane moment capacity (or their in-plane yield moment) exceeds their in-plane cracking moment.

3.14 Controlling Axial Load with Building Frame Systems

For most low- and medium-rise walls, axial load is beneficial because flexural strength normally increases with increasing axial load. However, as design axial load increases, the maximum permitted percentage of vertical (longitudinal) reinforcement decreases, which may begin to control the design (**Figure 3-5**). For walls whose design is controlled or limited by ρ_{max}, it can be advantageous to reduce the design axial load by detailing the wall so that it is subjected to lateral loads but not vertical loads from the diaphragms. This can be accomplished by detailing the wall so that abutting horizontal diaphragm elements are free to move vertically with respect to the wall but are restrained horizontally by the wall. If axial loads are limited consistently in all the wall elements in the structure, the SFRS may be classified as a "building frame system" per ASCE 7.

3.15 Additional Provisions in IBC and ASCE 7

The designer should be aware that Chapter 21 of the IBC, in referencing TMS 402, includes various modifications to the TMS requirements. A complete review of the modifications is beyond the scope of this Guide, but provisions that most affect the design of special masonry shear walls include lap splice definitions that are in general more liberal than TMS 402, and maximum bar sizes that are more restrictive. The Masonry Designers' Guide (TMS 2013c) addresses the subject in detail.

Similarly, ASCE 7 §14.4 also includes modifications to TMS 402 requirements, addressing details of coupling beams, shear keys, and other issues, including the prohibition of lap splices in plastic hinge zones; however, IBC §1613.1 explicitly excludes Chapter 14 of ASCE 7, in effect nullifying that ASCE 7 provisions in jurisdictions that have adopted the 2012 edition of IBC.

4. Building Analysis Guidance

4.1 Analysis Procedures

For seismic design, ASCE 7 permits three types of analysis procedures to determine structural displacements and design forces in structural elements: the Equivalent Lateral Force (ELF) analysis procedure, the Modal Response Spectrum (MRS) analysis procedure, and the Seismic Response History (SRH) analysis procedure. A brief overview of these procedures is given in the NEHRP Technical Brief *Seismic Design of Cast-in-Place Concrete Special Structural Walls and Coupling Beams* (NIST 2012). For the design of masonry buildings, ELF is most common. However, according to ASCE 7 §12.6, ELF is not permitted for structures exceeding two stories and having horizontal or vertical irregularities of certain types, including torsional, stiffness, and soft-story irregularities. For such structures, either MRS or SRH are permitted to be used. With the ELF and SRH procedures, either linear or nonlinear structural analysis is permitted. Most masonry structures are stiff and have a limited number of stories, making the application of linear or nonlinear SRH of limited value.

Even though linear elastic analysis is most commonly used in design, plastic limit analysis, which explicitly accounts for the plastic mechanism of a reinforced masonry wall system, is preferred for certain wall configurations to arrive at a design that is rational and does not result in unexpected failure modes. Plastic limit analysis is especially suited for coupled walls or perforated walls that have shear-critical components. It can often be performed with hand calculations. Nevertheless, if a structure is very complex or highly indeterminate, the final design should be checked with nonlinear analysis using a computer model.

For structures without horizontal irregularities, two-dimensional models are normally sufficient, and the lateral load-resisting systems in the two orthogonal directions can be considered independently. Nevertheless, the designer may encounter the case of a special reinforced masonry wall that forms part of two intersecting lateral load-resisting systems and is subjected to axial load because of seismic forces acting along either principal plan axis equal to or greater than 20 percent of the axial design strength of the wall. In that case, the most critical combined effect of seismic forces in any direction should be considered, as specified in ASCE 7 §12.5. This can be the situation for flanged walls in a tall building where the axial load demand because of seismic forces is significant. For this situation, ASCE 7 §12.5.3 provides two alternatives for analysis. The first is to perform two-dimensional analysis for each of the two orthogonal directions independently and determine the most critical combination of 100 percent of the forces for one direction and 30 percent of the forces for the other. The second is to perform three-dimensional response history analysis in which ground motions in the two orthogonal directions are applied simultaneously. However, when a building has torsional irregularity, out-of-plane offset irregularity, or nonparallel system irregularity as defined in ASCE 7 Table 12.3-1, a three-dimensional model must be used in the analysis according to ASCE 7 §12.7.3. With torsional irregularity, either the MRS or SRH procedure must be used. In such analysis, the in-plane stiffness characteristics of the floor and roof diaphragms and the dynamics of the diaphragms have to be accounted for. Diaphragms can be modeled by either shell elements or line elements using a grid or truss representation. However, concrete diaphragms can often be considered rigid in plane.

Masonry structural walls should also be designed for out-of-plane loads, which can in fact dictate the amount of vertical reinforcement for walls with significant story heights as discussed in Section 5. The out-of-plane seismic force to be used in the design is given in ASCE 7 §12.11.1, while the design and analysis procedure is prescribed in TMS 402 §9.3.5. Because masonry walls are slender for out-of-plane bending, the P-Δ effect is important. TMS 402 provides two methods to account for this: a second-order analysis, which requires iteration, or a moment magnifier. To be consistent with the equations provided in TMS 402, the out-of-plane seismic force should be idealized as a uniformly distributed load.

4.2 Modeling Considerations and Structural Idealization

The behavior of a box-shaped reinforced masonry wall structure can be most directly modeled with shell elements (Lepage and Sanchez 2012). The main advantage of shell elements is that they account for both shear and flexural deformations of wall segments and can also model the response of a wall segment to simultaneous in-plane and out-of-plane loads. Linear and nonlinear shells elements that are available in commercial programs to model the behavior of reinforced concrete shear walls can also be used for reinforced masonry.

The use of linear shell elements is relatively straightforward. A sufficiently fine mesh should be used to capture the shear and flexural behavior of an elastic wall. The designer should be aware that linear models usually overestimate the stiffness of cracked wall elements.

Nonlinear shell models are not generally appropriate for design. If used at all, they should be used with caution because they have many numerical analysis and material parameters whose meanings may not be apparent to users unfamiliar with nonlinear analysis. Furthermore, most nonlinear shell elements cannot capture the shear behavior of wall segments dominated by diagonal tension and may overpredict their strength and ductility. Results of nonlinear analyses can also be highly sensitive to the size of elements in the mesh: too coarse a mesh could overpredict the capacity of a wall; a very fine mesh could result in an overly brittle post-peak flexural behavior. Nonlinear models are best suited to applications where they can be calibrated and validated by experimental data.

In most situations, two-dimensional frame models are more than adequate for the analysis of masonry wall systems. Frame elements can incorporate both flexural and shear deformations. Masonry wall systems often have large panel zones connecting vertical and horizontal wall segments. Although these zones are not completely rigid and may even have cracks, they can be treated as rigid in a frame model. This increases the degree of rotational restraint at the ends of the affected wall segments and therefore decreases the shear-span-to-depth ratio $M_u/(V_u d_v)$ in each segment. Because of the requirement for capacity design for shear, this normally results in higher required shear capacities and in that sense is conservative for design.

To develop an appropriate frame model, the structural designer needs to determine the probable locations of critical moments and shear forces and model the vertical and horizontal wall segments containing these regions with suitable beam-column elements. Once those critical design actions have been calculated and the corresponding reinforcement for the wall segments has been determined, the rigid zones can then be appropriately designed and detailed to resist the forces corresponding to the capacities of those adjacent wall segments.

Frame models discussed here can be used for either linear or nonlinear analysis. Examples of frame models representing different reinforced masonry wall configurations are shown in **Figures 4-1** and **4-2**. Although the development of an appropriate frame model is relatively straightforward for cantilever-wall and coupled-wall systems, as shown in **Figure 4-1**, it may be less so for a perforated wall, especially one with an irregular arrangement of openings (see **Figure 4-2(b)**, for example). For a perforated wall, the designer must identify the vertical and horizontal wall segments (dark gray areas bounded by openings in **Figure 4-2**) for which critical moments and shear forces are to be determined. Then, the designer must consider the remaining regions as rigid panel zones.

Different possible idealizations must be considered so that the most critical condition can be identified for the frame model. For the wall shown in **Figure 4-2(a)**, for example, two possible modeling alternatives exist, as illustrated by the two parts of the figure. One is consistent with the assumption of a crack propagating from the lower left-hand corner of the lower right window and separating the middle wall segment from the panel underneath the window. The other assumes that the panel underneath the window remains intact. The first assumption results in more flexure-dominated behavior in the middle segment, and the second introduces a more shear-

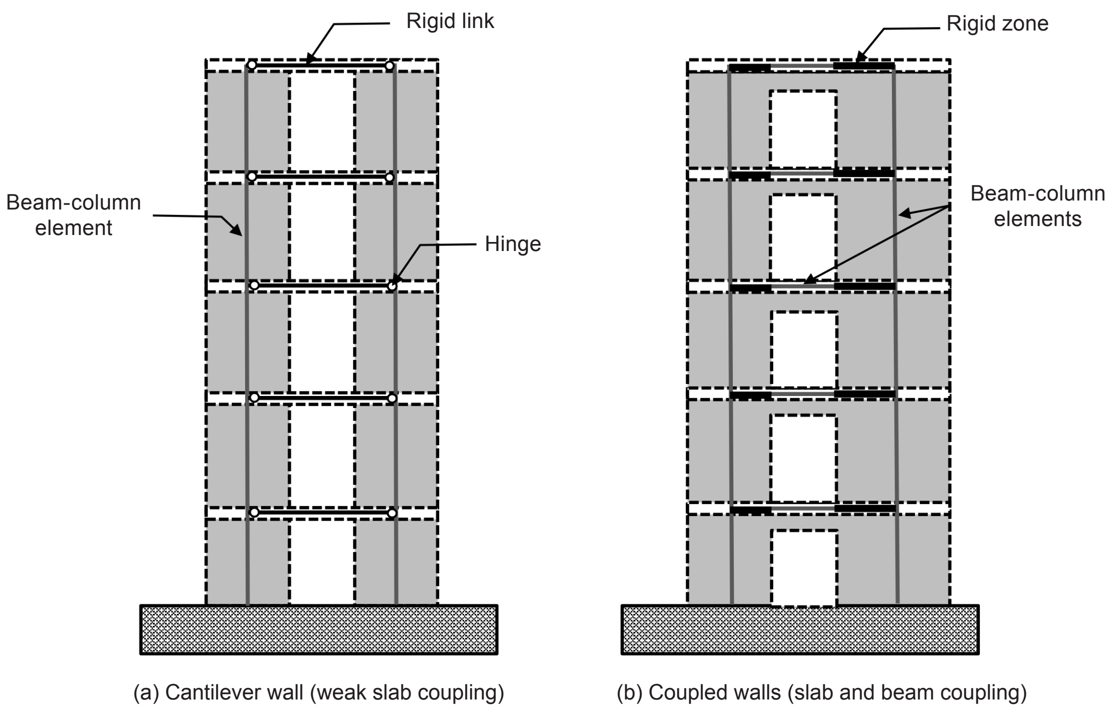
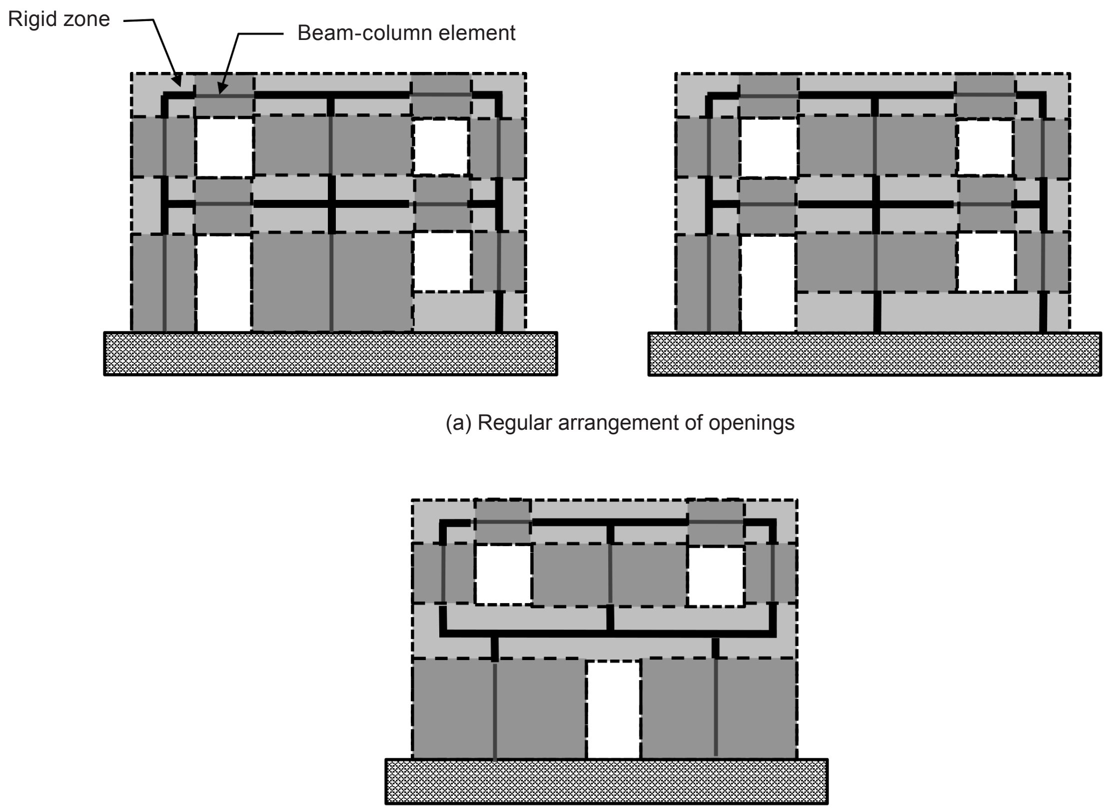


Figure 4-1. Frame models of cantilever and coupled walls.



(a) Regular arrangement of openings

(b) Irregular arrangement of openings

Figure 4-2. Frame models of perforated walls.

critical condition. However, it is likely that the actual wall behavior is somewhere in-between because the horizontal reinforcement beneath the window opening may prevent a complete separation of the panel by a crack.

For simple wall systems, a frame model can be analyzed by hand, although a computer analysis is usually cost-effective. For computer solutions, it is better to describe rigid zones by using kinematic constraints (i.e., slaving the degrees of freedom) rather than by using very large stiffness values, which may lead to inaccurate numerical results.

4.3 Elastic Analysis and Member Stiffness

Even with the SD method, linearly elastic structural models are most often used for the determination of design forces and moments for reinforced masonry wall systems. Furthermore, to check the story drift limit according to ASCE 7 §12.12.1, deflections are first calculated with linear elastic analysis, and the results are multiplied by an amplification factor, C_d, to account for the structural nonlinearity. For this purpose, good estimates of the elastic stiffness properties of wall elements in the structural model are important. To this end, shear as well as flexural deformations should be considered, although shear deformations are generally unimportant for wall segments with shear-span-to-depth ratio $M_u/(V_u d_v)$ greater than 2. Values of the elastic moduli and the shear moduli of clay and concrete masonry are prescribed by TMS 402 §4.2.2. For reinforced masonry, the models are required to incorporate the effects of cracking.

The effects of cracking are commonly addressed by using an effective moment of inertia, I_e, in place of the gross moment of inertia, I_g. Although TMS 402 does not provide guidance for this, ACI 318-11 §8.8.2 recommends that the effective moment of inertia for reinforced concrete walls be taken as 50 percent of the gross moment of inertia in general, which has been commonly assumed in practice for masonry walls, or as 70 percent of the gross moment of inertia for uncracked walls and 35 percent of the gross moment of inertia for cracked walls. However, the effective moment of inertia of a wall depends on many factors, such as the axial force level on the wall, the quantity of vertical reinforcement, the shear-span-to-depth ratio $M_u/(V_u d_v)$, and the slip of vertical bars in the foundation slab. Data from the wall tests of Shing et al. (1989), Sherman (2011), and Ahmadi (2012) suggest that the effective moment of inertia I_e for reinforced masonry walls with rectangular cross sections can be taken to be 15 percent of I_g. This value provides a good match with experimental results for fully grouted walls with shear-span-to-depth ratios ranging from 0.5 to 4.5, different levels of axial loads, and different vertical steel quantities, when the effective shear stiffness recommended below is used. For T-shaped flanged walls, limited data (He and Priestley 1992) have shown that

I_e can be 40 percent of I_g when the flange is in tension. The effective moment of inertia is smaller when the flange is in compression. However, the moment capacity of the section with the flange in compression is also less, and the decrease in flexural stiffness can reasonably be ignored when the moment capacity for that loading direction is relatively small. For I-shaped walls, I_e may also be assumed to be 40 percent of I_g as there is always a flange in tension during flexure. The behavior of flanged walls is discussed further in the following section. The recommendations of ACI for the stiffness of uncracked walls are not generally appropriate for masonry walls, and it is considered conservative to use the moment of inertia of the gross section, I_g, for uncracked walls.

The elastic shear stiffness of an uncracked wall segment can be calculated using the shear modulus of masonry and the effective shear area for the cross section using the principles of structural mechanics. However, the actual shear stiffness of a wall segment is often much lower than the theoretical value because of flexural and diagonal shear cracks, which may occur at a moderate lateral load. Experimental data (Shing et al. 1989) show that the secant shear stiffness of a reinforced masonry cantilever wall, with an aspect ratio of one, loaded to a point at which major diagonal cracks develop, can be as low as 20 percent of the theoretical value, and that of a wall loaded to 50 percent of its shear capacity can be about 50 percent of the theoretical value. Based on this observation, the effective elastic shear stiffness of a reinforced masonry wall can be taken as 35 percent of the theoretical value, which appears to be consistent with other wall test data as discussed in the above paragraph. For partially grouted walls, one may assume the cross-sectional area resisting shear to be the total cross-sectional area of the face shells of the masonry units plus the area of the grouted cells.

Under seismic actions, the beams or lintels in a masonry wall system deform in shear and flexure. It is difficult to structurally decouple a lintel from the vertical walls effectively, even with the use of control joints or a hinged design, because of diagonal strut actions that can develop in deep coupling elements. Hence, coupling elements should generally be designed to be structurally integrated with the walls, and their stiffness should be considered in analysis. In the absence of conclusive data, the effective moment of inertia and the effective shear stiffness of a reinforced masonry beam can be calculated with the same assumptions as those recommended for walls, even though the lack of axial forces in beams may lower the effective stiffness. For a masonry beam connected to a concrete slab with dowels, the composite action of the two needs to be considered and I_e can be assumed to be 40 percent of I_g. The effective width of the slab is discussed in the following section.

Once the stiffness of each wall segment has been determined, the resulting frame-element model can be analyzed, either by hand or by computer (depending on the level of complexity). For a cantilever wall system as shown in **Figure 4-1(a)**, the lateral seismic forces resisted by each wall can be assumed to be proportional to the lateral in-plane wall stiffness. However, if the diaphragms are flexible, the share of the seismic forces should be proportional to the tributary floor and roof areas for each wall. For the analysis of perforated walls, such as those shown in **Figure 4-2**, computer models may be more convenient.

4.4 Effective Widths of Wall Flanges and Coupling Slabs

When a flanged wall is subjected to flexure and shear, in-plane vertical stresses in the flange diminish as the distance from the web increases, an effect commonly known as “shear lag.” The portion of the flange farther away from the web is thus less effective in resisting flexure. To account for this, one can define an effective flange width over which the normal stress is assumed to be uniform. The effective flange width depends on many factors, such as the shear-span-to-depth ratio of the wall, the thickness of the flange, and the presence or absence of cracks in the flange. The effective width increases with the shear-span-to-depth ratio. The effective width changes when the inelastic deformation of the wall increases, and it also depends on whether the flange is in tension or compression. The shear-lag effects are less significant when the flange is in tension because of cracking. Both the stiffness and the strength of a T-section wall are expected to be higher when the flange is in tension than when it is in compression.

According to TMS 402 §5.1.1.2.3, the flange width that is effective on each side of the web should be taken as 6 times the nominal flange thickness when the flange is in compression and 0.75 times the floor-to-floor wall height when the flange is in tension, but should not exceed the actual width of the flange. This is a conservative estimate for determining the flexural strength of a flanged wall, and the actual contribution of the flange could be higher. However, when the flange is in compression, the flexural strength is insensitive to the assumed effective width because it results in only a small shift of the neutral axis of bending. Hence, in analysis, this distinction is not important and one can assume that the effective flange width on each side of the web is 0.75 times the wall height. However, to protect flanged walls from shear failures and from severe toe crushing in the web, which may happen in a T-section wall, increasing the above-recommended effective flange width by a factor of 1.5 when doing shear capacity design and ductility checks is prudent.

In coupled reinforced masonry walls, the coupling elements can be concrete slabs alone or concrete slabs plus reinforced masonry lintels, which are often connected by dowels. In either case, it is not necessarily true that the entire tributary width of the slab is fully engaged in the coupling action. A study by Seible et al. (1991) concludes that for the slab alone, the effective width of the slab that contributes to the coupling moment on either side of the wall can be taken to be equal to the width of the door opening, which in their case was 40 inches. Slabs considered in that study were side-by-side, 6 inches thick, 40 inches wide, precast hollow-core planks with cast-in-place concrete topping 2 inches thick. The planks ran parallel to the walls. In the first of two test specimens in that study, the small concrete element between the planks above the door opening had four longitudinal bars but no transverse reinforcement; in the second test specimen, the small concrete element had a large amount of transverse reinforcement and was attached to a reinforced masonry lintel that was not structurally connected to the walls. The first specimen experienced a brittle shear failure in the coupling slab, which led to a sudden drop in the lateral resistance of the wall system. The second specimen, in contrast, showed ductile coupling behavior. No data are available for slabs spanning openings wider than 40 inches.

Based on the above information, when the walls are coupled by slabs alone, the effective bending width of the slab on either side of the wall can be assumed to be 40 inches but not greater than the width of the precast plank next to the wall regardless of the width of the opening, provided that a ductile coupling behavior can be expected for the slab. Otherwise, coupling effects should be ignored. A 40-inch effective width can also be assumed for cast-in-place reinforced concrete slabs. Because there are no data on the effective width of a slab with planks perpendicular to the walls, the coupling effects of such slabs should be ignored because tension cannot be transmitted across the planks.

When the concrete slab is connected to a reinforced masonry lintel with dowels, the result is a composite T-section or L-section. For reinforced concrete T-beams, ACI 318 specifies that the effective width of overhanging flange on either side of a web should not be taken greater than 8 times the slab thickness or 1/2 of the clear distance to the next web with the total effective flange width not to exceed 1/4 of the span length of the beam. For the test shown in **Figure 3-8**, the effective width of the slabs above the door openings on each side of the wall was estimated to be far beyond the door openings, which were 40 inches wide. In view of this, when a concrete slab is connected to a reinforced masonry lintel with dowels, it is recommended that the effective slab width on each side of the wall be taken as at least 6 times the slab thickness. Nevertheless, if the composite coupling beam is not as strong as the walls, then the effect of the lintel should be ignored because it cannot develop the ductility required of a coupling beam as discussed in Section 3. When precast planks are perpendicular to the walls, only the reinforced concrete topping should be considered effective in developing tension. For capacity design checks to avoid brittle shear behavior in walls, the effective slab widths recommended above should be increased by a factor of 1.5.

4.5 Plastic Limit Analysis

Plastic limit analysis is more suitable than elastic analysis for some configurations of masonry walls. It is also an essential tool for the Limit Design method in Appendix C of the TMS 402. For limit analysis, one needs to identify the plastic mechanism that is developed by the wall system when it reaches the plastic limit state. A fundamental assumption for such analysis is that the plastic hinges identified are able to sustain inelastic deformations associated with the mechanism without strength degradation, thus allowing the redistribution of resisting actions. Hence, for walls whose design actions are determined using plastic limit analysis, either the masonry elements must be detailed for the inelastic deformation capacities consistent with the anticipated inelastic deformations, or the wall system must be designed so that the inelastic deformation demand on each masonry element is less than or equal to the corresponding inelastic deformation capacity. The designer must make sure that such conditions are met. Following the Limit Design requirements stipulated in Appendix C of TMS 402, as discussed in Sections 3 and 5 of this Guide, is one way to meet these conditions. In this section, two examples of plastic limit analysis are presented, one for coupled walls and the other for perforated walls.

For coupled walls, which have plastic hinges developed in the coupling elements, elastic analysis may overestimate the coupling effects and underestimate the moment demands on the walls. Therefore, plastic limit analysis is the preferred method to determine design forces for coupled-wall systems. It can also result in a more efficient design process by directly accounting for load redistribution because of the coupling actions. The first step in limit analysis is to identify a kinematically admissible plastic mechanism (that is, a mechanism that maintains displacement continuity among wall segments) with the locations of plastic hinges consistent with that mechanism identified. The internal distribution of design actions for each wall segment can then be determined by equilibrium.

Consider, for example, the wall system shown in **Figure 4-1(b)**, for which the plastic mechanism shown in **Figure 4-3** can be identified. If the floor slabs and lintels (if applicable) have already been designed for gravity loads, the plastic moment capacities, M_{pi}^{b}, and the corresponding shear forces, V_{M}^{b}, in each coupling element can be calculated. Then, considering the equilibrium of axial forces for each wall, one can determine the tension force, T^{w}, or compression force, C^{w}, introduced at the base of each wall by the coupling elements.

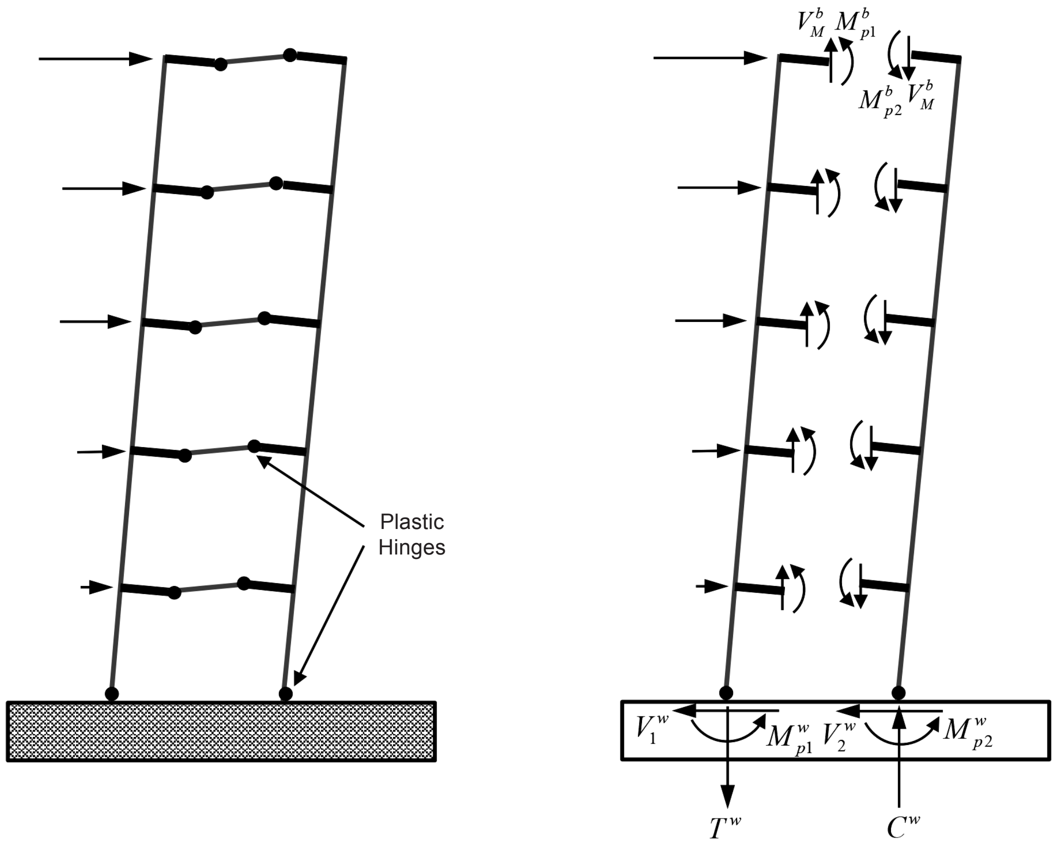


Figure 4-3. Plastic mechanism of a coupled-wall system.

The total overturning moment demand depends on the design seismic forces, determined according to ASCE 7, and must be equaled or exceeded by the overturning moment capacity associated with the mechanism. The designer has several options to distribute the plastic moment resistance between the walls to achieve this objective. If the designer chooses to use equal percentages of vertical (longitudinal) reinforcement for each wall and if the axial forces from overturning are neglected, the moment capacity of each wall is more or less proportional to the square of its plan length. If, in contrast, the designer chooses to have the flexural strength of each wall be proportional to its flexural stiffness, the moment capacity of each wall is more or less proportional to the cube of its plan length. For these or other options, the designer can determine the required moment capacities for each wall. In this step, the axial forces caused by lateral seismic forces may be ignored. The reason is that the decrease of moment capacity of one wall, caused by a reduction in axial compression, is offset by the increase of moment capacity of the other wall, caused by an increase in axial compression.

Once that initial design has been completed, the designer should check the overturning moment capacity, including the effects of axial forces from seismic overturning. Once the moment capacity of each wall is verified, its lateral seismic forces can be determined by equilibrium, and a capacity design for shear can be completed. For coupled-wall systems that are not symmetrical, loadings in both in-plane directions should be considered, and the more critical direction should be considered for design.

The application of plastic limit analysis to perforated wall systems is illustrated by the wall with the overlaid frame model shown on the left in **Figure 4-2(a)**. Its plastic mechanism is shown in **Figure 4-4**. In this case, the total lateral load resistance is governed by the plastic-hinge capacities of the three piers (vertical wall segments), which can be either flexure-dominated or shear-dominated. The distribution of lateral resistance among the three piers is a design decision. If the designer chooses to have the lateral load resisted by each pier in proportion to its lateral stiffness (elastic behavior), then the right pier will resist most of the lateral load because of its smaller height. Because this pier is also the most vulnerable to diagonal shear failure, this design decision may result in the failure of the right pier in shear and in the formation of an undesirable story mechanism involving the remaining piers. To guard against this behavior, the flexural capacities of each pier should be adjusted so that the left and middle piers resist as much of the lateral forces as possible. As before, those flexural capacities are initially calculated without the consideration of axial loads caused by overturning moment and then checked with them included.

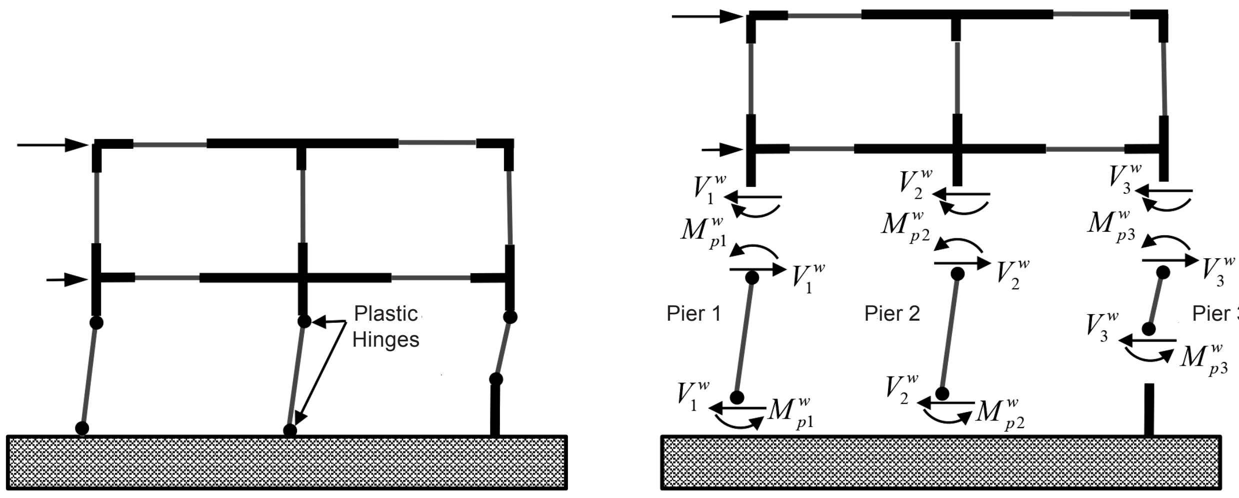


Figure 4-4. Plastic mechanism of a perforated wall (element axial forces are not shown for clarity).

As before, for unsymmetrical configurations, loadings in both in-plane directions should be considered, and the more critical direction should be considered for design.

After the quantity of vertical reinforcement in each pier has been determined, the designer has to determine the effects of the axial forces introduced by overturning on the load and deformation capacities of each pier. This evaluation can be done by hand calculations or a detailed nonlinear analysis, depending on the complexity of the wall system. This check is recommended because the additional axial forces can have a significant influence on the shear capacity of a pier. In fact, Limit Design requires that these axial forces be considered in determining the deformation capacity of plastic hinges but not for strength checks. Limit Design further requires that the maximum shear developed in shear-dominated wall segments be no greater than one-half of the shear strength determined in accordance with TMS 402 §9.3.4.1.2. It is recommended here, however, that the contribution of masonry to shear capacity be ignored for shear-dominated wall segments subjected to tension. An iterative process may be required to arrive at a distribution of shear resistances among wall segments that is consistent with the axial forces in those wall segments at the formation of a mechanism.

For detailed nonlinear analysis, beam-column elements with a plastic hinging capability should be used. Beam models with fiber sections, which account for the axial load-moment interaction, can be used for such analysis and for determining the rotation capacity of a plastic hinge. However, these models do not account for shear-dominated behavior. Hence, if a pier is expected to be shear-critical, then its moment capacity should be specified in the model so that the maximum shear force developed does not exceed the maximum recommended above. A shear-dominated pier is brittle and cannot sustain the same deformation as a flexure-dominated pier, and as a consequence, a large portion of its shear resistance could be lost before the flexural capacities of the other piers have been reached. Hence, one should be conservative in estimating the contribution of shear-dominated piers to a line of resistance in plastic analysis.

5. Design Guidance

Design guidance in this section is based on Chapter 9, Strength Design of Masonry, of TMS 402 and all applicable chapters of TMS 602, Specifications for Masonry Structures and the companion commentaries. Following that, Appendix C, Limit Design Method, and Chapter 8, Allowable Stress Design of Masonry, are briefly addressed.

5.1 Load and Resistance Factors

ASCE 7 Chapter 12 defines the factored load combinations applicable to reinforced masonry shear wall design. The strength reduction factors, ϕ, are specified in Chapter 9 of TMS 402.

5.2 Considerations for Preliminary Estimates of Design Base Shear

Design of masonry walls requires that trial configurations of masonry shear walls in plan and elevation be proposed and that their lateral resistance be compared with their seismic base shear demand. This process is continued iteratively until a satisfactory wall configuration is achieved.

Although the computation of seismic base shear demand is precisely described in ASCE 7 Chapter 12 and the computation of base lateral resistance is precisely described in Chapter 8 (ASD), Chapter 9 (SD), and Appendix C (Limit Design) of TMS 402, particular aspects of those documents can influence the choices that a structural designer can make in the above iterative process.

Base shear demand is increased by several mandatory amplification factors:

1. The redundancy factor, ρ. An increase in base shear demand because of a non-redundant wall layout can be avoided by meeting the conditions of ASCE 7 §12.3.4.2.

2. Inherent and accidental torsion can increase seismic base shear demand by a factor varying from 1.2 to 1.5. This can be minimized by avoiding plan eccentricities between the center of mass and the center of rigidity at each level, and by increasing torsional stiffness. This is done by arranging planar walls symmetrically in plan (assuming a uniform distribution of mass in plan), far from the center of rigidity, or by locating walls that form closed tubes in plan concentrically with the center of rigidity.

3. When the seismic base shear demand is resisted by multiple parallel walls having different configurations of openings and wall segments, the seismic shear demand at each level is not distributed uniformly among wall segments. If the seismic demand is distributed according to the elastic stiffness of wall segments, design actions depend on the dimensions of each segment, and these actions can vary widely from segment to segment. If a plastic distribution among wall segments is assumed (Limit Design), the designer has more control, but design actions can still vary considerably from segment to segment.

4. The ASCE 7 §2.3 requirement that multiple load combinations be considered may result in flexural overstrength in wall segments for certain load combinations when the flexural reinforcement in the wall segments is governed by a different load. This flexural overstrength can result in increased shear demand for wall segments in shear capacity design checks.

5. The seismic response of slender multi-story buildings may be influenced by the participation of perpendicular wall elements with respect to the line of seismic action, which may or may not be part of the SFRS. This participation may increase the flexural capacity and thus the base shear demand on walls in the line of seismic action.

5.3 Lateral Load Distribution and Lateral Stiffness

The seismic base shear demand must be distributed among the seismic load-resisting elements of a building structure according to their stiffnesses (using Chapter 7 of TMS 402) or according to their strengths (using Appendix C of TMS 402 or by other means). Although a structure may be designed so that seismic resistance is provided by shear walls, seismic resistance may also be provided by columns and piers (vertical wall segments).

For structures assigned to SDC C and higher, TMS 402 §7.4.3.2.4 requires a SFRS in which at least 80 percent of the stiffness of each line of resistance in each story must be provided by seismic force-resisting walls. When a response modification factor, R, not greater than 1.5 is used, TMS 402 permits the inclusion of columns and piers in the SFRS.

5.4 General Approach to Strength Design of Shear Walls

Once design forces for each wall segment have been determined, the sequence of design of vertical (longitudinal) and horizontal (transverse) reinforcement for each segment is as follows. See **Figure 1-1**.

1. Design the wall segment for the axial load and out-of-plane wind and seismic loads.
2. Design for shrinkage, permanent moisture expansion, and thermal movements.
3. Check prescriptive reinforcement requirements for special walls.
4. Design the wall segment for the axial and in-plane seismic loads.
5. Check cracking moment (optional).
6. Check shear capacity.
7. Check sliding shear capacity (optional).
8. Check maximum reinforcement limits.
9. Check wall behavior mode.
10. If necessary, check repercussions of shear-dominated behavior on other components.

5.4.1 Design for Out-of-Plane Forces

Most reinforced masonry walls must resist a combination of lateral forces (both in-plane and out-of-plane) and axial forces. In many cases, the out-of-plane forces control the design. For example, the reinforcement required for out-of-plane wind or seismic forces on exterior walls 16 to 20 feet tall often meets or exceeds the minimum prescriptive requirements for a special shear wall. For this reason, it is generally advisable to perform the out-of-plane design first and then check the resulting vertical reinforcement for the in-plane loads.

There is also interplay between out-of-plane and in-plane designs. It is possible that the reinforcement required for the out-of-plane load may drive an otherwise flexure-dominated wall into a shear-dominated failure mode.

Wall reinforcement is designed to resist out-of-plane forces as required by TMS 402 §9.3.5, for the combination of factored design axial load, P_u, and the factored out-of-plane design moment, $M_{u,0}$, which is magnified for out-of-plane P-Δ effects. For out-of-plane loading, walls typically have ratios of $M_u/(V_u d_v)$ far in excess of unity and are flexure-dominated. Because their out-of-plane ductility demand is low, the maximum permissible reinforcement percentage as governed by the out-of-plane demand is calculated using an α factor of 1.5 in accordance with TMS 402 §9.3.3.5.1.

Out-of-plane deflection limits under unfactored service loads are stipulated in TMS 402 §9.3.5.5. These limits are intended to ensure elastic behavior under out-of-plane loads so that the wall can be anticipated to return to its original planar shape and plumbness after the out-of-plane lateral load is removed.

5.4.2 Design for Shrinkage, Permanent Moisture Expansion, and Thermal Movements

Over time, concrete masonry undergoes long-term drying shrinkage, and clay masonry undergoes long-term permanent moisture expansion. All masonry undergoes reversible thermal deformations caused by temperature changes. If these deformations are not accommodated by shrinkage (control) joints in concrete masonry, or by expansion joints in clay masonry, they produce stresses in the masonry. When tensile stresses caused by a combination of restrained thermal deformations and moisture-related deformations, exceed the tensile strength of the masonry, it cracks. Although this cracking is not structurally important in reinforced masonry, it can be unsightly. To control the locations and widths of cracks, the design of walls must incorporate combinations of vertically oriented movement joints and horizontal reinforcement.

These vertically oriented movement joints define the vertical boundaries of vertically oriented wall segments. Depending on how they are detailed, control joints may define the structural configurations and aspect ratios of wall segments, and may dictate the transverse (usually horizontal) reinforcement requirements of special walls. For this reason, design for thermal and moisture-related movements (and the consequent location of movement joints) must precede the design for in-plane forces. NCMA (2010) and BIA (2014) give guidance on the use of movement joints and reinforcement to control crack locations and widths. Horizontal reinforcement can consist either of deformed reinforcement placed in grouted bond beams or of ladder-type joint reinforcement placed in bed joints. The latter is preferable for partially grouted masonry because of the cost associated with installing closely spaced bond beams. The vertical spacing of bond beams effectively limits grout pour height, and the use of joint reinforcement allows more economical pour heights.

5.4.3 Check Prescriptive Reinforcement Requirements for Special Walls

Reinforcement must meet the prescriptive requirements of TMS 402 §7.3.2.6 for special reinforced masonry shear walls and the applicable sections of TMS 402 §7.4, SDC requirements. The requirements for special walls are illustrated graphically for one possible wall configuration in **Figure 5-1**.

The intent of the prescriptive requirements is to ensure a reasonable amount of ductility. Regardless of prescriptive detailing requirements, however, available ductility may be limited by the wall configuration and the consequent aspect ratios of critical wall segments.

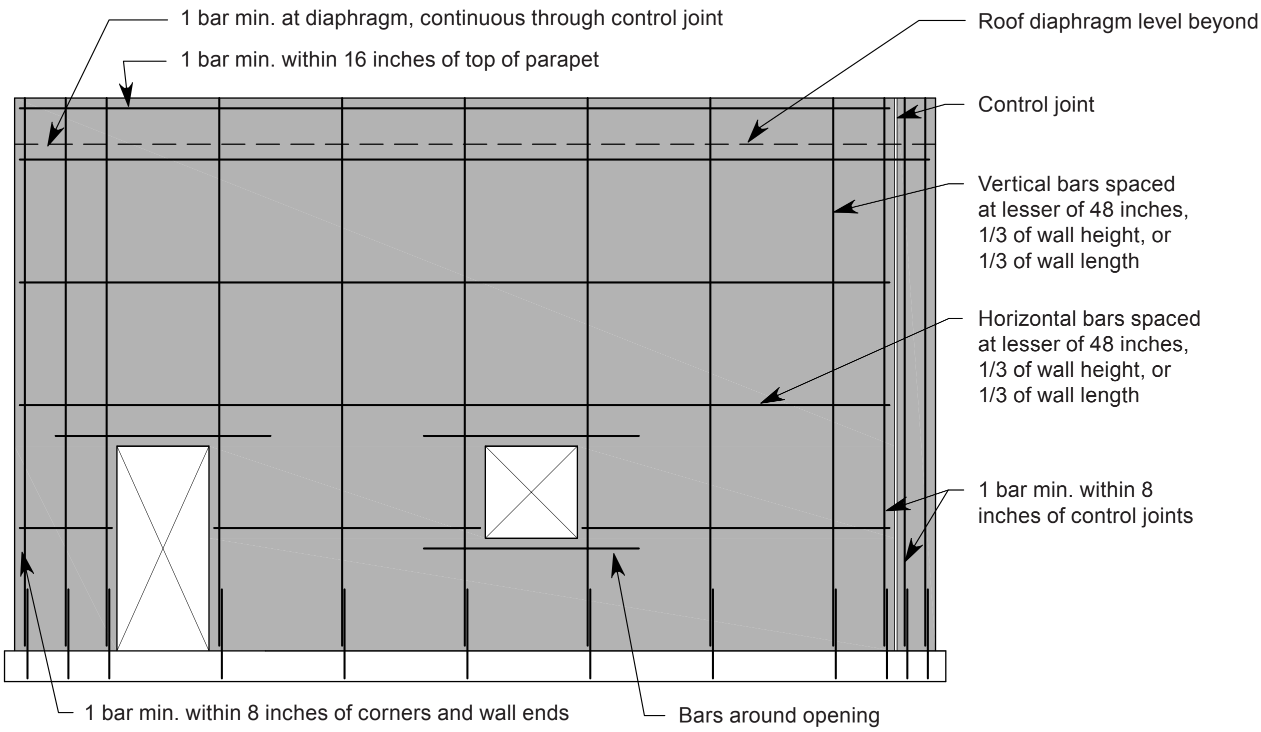


Figure 5-1. Conceptual representation of minimum reinforcement requirements for reinforced masonry special walls.

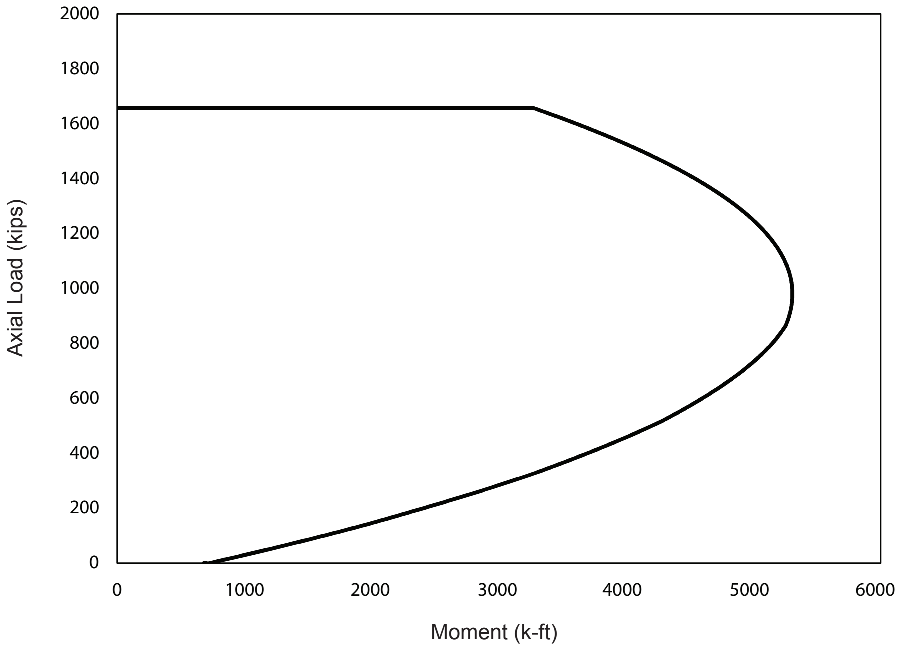


Figure 5-2. An example moment-axial load interaction curve for a concrete masonry shear wall.

5.4.4 Check In-Plane Flexural Capacity in the Presence of Axial Loads

At this point in the design process, the structural designer knows the geometry of the wall segments, has calculated the required load combinations, and has preliminary layouts of vertical (longitudinal) and horizontal (transverse) reinforcement for each segment, including any joint reinforcement used to control crack widths. The next step is to design the wall for in-plane forces as required by TMS 402 §9.3.6 for the combination of factored design axial load, P_u, and the factored design moment, M_u. This is typically done by developing a moment-axial load interaction diagram, as illustrated in **Figure 5-2**, to compare the design in-plane flexural capacity of each segment with its factored strength demands for the various load combinations that are plotted as points on the diagram. Finally, the designer must adjust flexural (vertical) reinforcement as necessary.

In contrast to design for out-of-plane forces, no moment magnifier is used; instead, by TMS 402 §9.3.4.1.1, the nominal axial capacity is not permitted to exceed a value that depends on the laterally unsupported length and the weak-axis radius of gyration of the wall cross section.

5.4.5 Check Cracking Moment

Although it is not a requirement of TMS 402, some designers prefer that walls have sufficient vertical reinforcement that their nominal in-plane moment capacity (or their in-plane yield moment) exceeds their in-plane cracking moment. In this case, the cracking moment is calculated using the modulus of rupture specified in TMS 402 §9.1.9.2, and the reinforcement is adjusted accordingly.

5.4.6 Check Shear Capacity and Sliding Shear Capacity

The shear capacity of the wall is checked in accordance with TMS 402 §9.3.4.1.2, which sums separate contributions to shear resistance from masonry and from reinforcement. Shear capacity is reduced for partial grouting. Nominal shear capacity is capped at a limiting upper value, beyond which additional shear reinforcement is considered ineffective in increasing the shear capacity of the wall.

Shear reinforcement can consist of deformed reinforcement in grouted bond beams or wire reinforcement in bed joints. Shear reinforcement in bond beams is required to be developed in accordance with TMS 402 §7.3.2.6(d). Requirements on wire joint reinforcement used as shear reinforcement are noted in TMS 402 §9.3.3.7 and are discussed further in Section 7 of this Guide.

Special masonry walls are also required to satisfy the shear capacity design requirements of TMS 402 §7.3.2.6.1.1. When designing special reinforced masonry shear walls to resist in-plane forces in accordance with TMS 402 §9.3, the design shear strength, ϕV_n, is required to exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the element, except that the nominal shear strength, V_n, need not exceed 2.5 times required shear strength, V_u.

Sliding shear capacity is not addressed by current TMS 402 provisions dealing with concrete masonry or clay masonry. Although designers sometimes rely on ACI 318 or FEMA 306 (FEMA 1999) to estimate sliding shear capacity, experimental evidence suggests that those references may be unconservative for walls with shear-span-to-depth ratios greater than unity (Murcia-Delso and Shing 2012; Morrison and Bennett 2013). The sidebar on shear-friction strength shows a method that is being considered for inclusion in TMS 402 to calculate sliding shear capacity. For sliding shear checks, the designer must determine the shear-friction strength that is needed to prevent the sliding of a wall along its base. For this purpose, a ϕ factor of 0.80, the same as that for shear, can be used. The sliding shear strength need not exceed the shear demand that corresponds to 1.25 times the nominal flexural strength of the wall or the nominal shear strength of the wall. For low-rise structures with long walls, there is often significant overstrength in flexure and in shear, and the ultimate limit state is most likely base sliding. For this situation, it may be prudent to use a base shear demand calculated with an R factor of 1 if base sliding is to be prevented in the design earthquake.

Shear-Friction Strength

Shear friction is a limit state not addressed in TMS 402 but is under consideration. According to the current proposal, the nominal shear-friction strength, V_{nf}, at the sliding-critical interface of a wall can be calculated using the following formula:

$$V_{nf} = \mu(\gamma_f A_s f_y + P_u) \geq 0$$

in which P_u is the factored axial load, which is negative when a wall is in tension, A_s is the total area of vertical steel crossing the shear plane, and f_y is the nominal yield strength of the vertical steel. Unless case-specific data are available, the coefficient of friction μ can be taken as 0.65. The reduction factor γ_f is to be calculated as follows:

$$\gamma_f = 1\text{-}2\chi$$

For $M_u/(V_u d_v) \geq 1.0$, χ is the ratio of the area of vertical reinforcement in compression to the total area of vertical reinforcement, A_s, crossing the shear plane calculated at the nominal moment capacity of the wall under axial load P_u. For $M_u/(V_u d_v) \leq 0.5$, χ is to be 0. The value of χ can be linearly interpolated between 0 and the ratio of the compression steel as defined above for $M_u/(V_u d_v)$ between 0.5 and 1.0.

For walls with uniformly distributed vertical reinforcement and $M_u/(V_u d_v) \geq 1.0$, the following equation can be used to estimate the value of χ:

$$\chi = \frac{c}{d_v} \leq 1.0$$

in which c is the distance of the neutral axis from the extreme compression fiber calculated at the nominal moment capacity of the section.

This will be at the discretion of the designer. When shear keys are used at the wall base, sliding shear resistance can be significantly increased well beyond that calculated with the method provided in the sidebar.

5.4.7 Check Maximum Reinforcement Limits

The designer must now confirm that the percentages of vertical (longitudinal) reinforcement do not exceed the maximum permissible percentages given by TMS 402 §9.3.3.5, using the factor $\alpha = 4$ for special reinforced masonry shear walls loaded in-plane, and the factor $\alpha = 1.5$ for walls loaded out-of-plane. The commentary to this code section has many helpful equations for different wall characteristics (full versus partial grouting, concentrated versus distributed reinforcement, and in-plane versus out-of-plane loading).

Alternatively, the boundary element provisions of TMS 402 may also be used. Even if boundary elements are not incorporated in the final design, the alternative triggers of those provisions potentially allow an increase in the reinforcement beyond ρ_{max} as long as certain requirements in TMS 402 §9.3.6.5 are met. Maximum reinforcement requirements may also be circumvented for selected lines of resistance by invoking the Limit Design procedure in Appendix C.

5.4.8 Check Wall Behavior Mode

At this point in the design process, the designer should establish whether the special wall is flexure-dominated (the implicit code intent for special walls) or shear-dominated. When a wall is shear-dominated, it may be possible to achieve flexure-dominated behavior through adjustment of reinforcement or wall geometry. Several design options are available:

1. Increase the shear reinforcement if possible. Above the cap on V_n, however, further increases in horizontal (transverse) reinforcement are not permitted to be considered as increasing the shear capacity of the wall.
2. Reduce the vertical (longitudinal) reinforcement to the minimum required to meet both factored moment demand and the prescriptive minimum reinforcement requirements.
3. Increase shear strength by fully grouting the wall.
4. Increase the thickness of the wall to increase its shear strength or adjust the wall's height and/or plan length to increase its aspect ratio, although architectural considerations may constrain adjustments to the basic wall geometry.

If none of these options is successful, the designer must accept the prospect of shear-dominated behavior and may consider reanalyzing the structure with a lower response modification factor (see sidebar).

Design of Shear-dominated Walls

It is the implicit intent of ASCE 7 and TMS 402 that special walls be flexure-dominated, with behavior that justifies the response modification factor R=5 assigned to them. However, as discussed in Section 3, walls designed in accordance with the provisions of these codes may still be shear-dominated. There is little direct guidance in TMS 402 to address this situation, which raises concerns that require the engineering judgment of the designer. The first is the possibility of brittle shear failure in elements of the gravity load-bearing system, albeit at lateral loads that may be much higher than the code-required design loads. This Guide does not address the complexity or repercussions of this possibility, nor does it advocate one approach or another to address it, because too many scenarios are possible.

Another implication of shear-dominated behavior in wall elements is the understanding that such elements may behave nearly elastically, attracting forces much higher than those calculated using a response modification factor R=5. This has implications for both the wall element itself and for other elements of the seismic load path that will have correspondingly increased demand. Such elements may include diaphragms, diaphragm chords, connections of diaphragms to shear walls, and collectors.

One approach available to the designer would be to design all elements of the seismic lateral load-resisting system associated with shear-dominated walls using a response modification factor less than the code value for special reinforced masonry walls (for example, R =1.5).

5.5 Limit Design Method

The Limit Design method is new to the 2013 edition of TMS 402 and set forth in Appendix C of that document. It is intended for use on specific, problematic lines of resistance within structural systems designed in accordance with the SD provisions of Chapter 9. "Problematic lines of resistance" are usually perforated walls with shear-dominated elements or coupled walls that appear to have overloaded coupling beams or undesirable distribution of forces between elements resulting in challenging local reinforcement requirements. The process follows these basic steps:

1. Perform an ASCE 7-compliant seismic analysis of the building per Section 4 of this Guide. Elastic analysis is generally suitable. From this analysis, determine the following:

   (a) structural element forces

   (b) inelastic displacement of the roof at each line of resistance. In general, it is satisfactory to estimate these values by first performing an elastic analysis using cracked section stiffness based on 50 percent of gross cross-section properties or the recommendations in Section 4.3 of this Guide and then amplifying the resulting displacements with the appropriate C_d factor given in ASCE 7.

2. Design lines of resistance using conventional SD procedures as described previously in this section.

3. Where problematic lines of resistance are identified, invoke the alternative Limit Design method in Appendix C for those lines of resistance only.

4. Begin with an assumed amount and distribution of reinforcement, preferably one associated with the minimum flexural reinforcement required by TMS 402.

5. Determine the governing yield mechanism for these lines of resistance when subjected to the loading patterns associated with the maximum base shear from the ASCE 7 analysis. There may be multiple kinematically admissible mechanisms, but the mechanism with the lowest-energy or resistance governs. Use the nominal capacity M_n of each masonry element as its hinge capacity.

6. Determine whether any elements are shear-dominated:

   (a) Determine the nominal shear strength of each element, V_n, according to TMS 402 §9.3.4.1.2.

   (b) If the shear strength of an element is less than twice that required to develop the moment capacity, M_n, of the element, consider the element to be shear-dominated for purposes of the Limit Design method.

   (c) Where shear-dominated elements exist, if possible, reduce the hinge capacity M_n or increase the shear strength until the shear strength of the element is more than double that required to develop M_n of the element.

   (d) Where shear-dominated elements cannot be avoided, limit the plastic hinge strength to the moment associated with one-half of the nominal shear strength, V_n .

7. Ensure that the mechanism strength, reduced by a strength reduction factor, ϕ =0.8, is greater than the base shear demand on the line of resistance as determined by the ASCE 7 analysis method.

8. Ensure that hinge rotation capacities are in excess of rotation demand associated with the design drift (determined in 1(b) above) imposed on the governing mechanism. Note that shear-dominated elements must be checked against more stringent deformation limits as specified in Appendix C.

9. Design non-yielding components (including connection regions) in the line of resistance to ensure that they remain elastic.

5.6 Allowable Stress Design

Thus far in this Guide, discussion of the design of reinforced masonry has focused entirely on modern SD methods. However, the ASD provisions of TMS 402 Chapter 8 and IBC §2107 are still widely used, owing to their simplicity and their applicability to a wide variety of masonry configurations and loading conditions. In many circumstances, they provide engineers a design alternative to the more prescriptively constraining ductility provisions of SD, while producing generally comparable structural capacity.

In practice, ASD entails comparing calculated stresses produced by, unfactored loads in accordance with ASCE 7 load combinations, to allowable stresses. As a result of this approach, the designer evaluates the structure under service level loading, and corresponding conditions of stress, that more closely align with those encountered in service than the factored loads associated with SD, arguably providing better insights into the performance of the structure when subjected to routine loading. On the other hand, this design approach provides little information regarding the ultimate strength and deformation capacity of the constituent components. ASD is a useful design method that has produced innumerable, well-performing masonry designs and can continue to do so. However, because of the absence of some of the specific ductility constraints of the SD method, the designer is encouraged to use engineering judgment to decide whether elements designed by ASD, and particularly elements with larger percentages of longitudinal reinforcement than permitted by SD, have sufficient inelastic deformation capacity.

6. Additional Design Requirements

Additional design requirements for masonry are covered at length in the references to this Guide, particularly in the *Masonry Designers' Guide* (TMS 2013c). Some of these requirements are highlighted below.

6.1 Quality Assurance

The provisions of TMS 402 are based on a presumption of good-quality construction. Therefore, Chapter 3 of TMS 402 (design requirements) mandates the existence of a quality assurance program meeting specific requirements, and it references TMS 602. Article 1.6 of TMS 602 (construction requirements) mandates a quality assurance program whose specific requirements are identical to those of TMS 402. In this way, the combination of TMS 402 and TMS 602 mandates an identical quality assurance framework for the designer and the contractor.

The quality assurance programs linked in TMS 402 and TMS 602 are tied to the Risk Category as defined in ASCE 7 or to the legally adopted building code.

6.2 Masonry Materials

The selection of masonry units, mortar, grout, and reinforcement by a designer must comply with the ASTM International (ASTM) specifications referenced by TMS 602. Within those bounds, materials that are locally available and consistent with local construction practice are generally the most economical.

6.2.1 Mortar

The designer is generally advised to use the lowest strength mortar that satisfies the job requirements (Type S or N mortar). Higher-strength mortar (Type M) can be difficult to work with. For fully grouted assemblies, mortar strength has little influence on the compressive strength of masonry. For partially grouted assemblies, mortar strength can have a greater effect.

There are two permitted methods for varying compliance with the specified compressive strength of masonry. The unit strength method is based on ungrouted assemblies, and it correlates moderately to mortar strength. The prism test method, ASTM C1314 Prism Test Method (ASTM 2012), more accurately accounts for the strength of the grout as well as the mortar.

6.2.2 Grout

In a reinforced masonry assembly, grout bonds deformed reinforcement to masonry, protects that reinforcement from corrosion, and increases the masonry cross section. The interaction of the grout with the surrounding units can be complex, but for the purposes of design, it is generally assumed that the units and grout behave together homogeneously with the properties of the assembly. Grout must generally meet the requirements of ASTM C476 (ASTM 2010), either by proportion or by compressive strength). Grout meeting the proportion requirements can be assumed to have a compressive strength of at least 2,000 psi. If f'_m exceeds 2000 psi, the compressive strength of the grout must equal or exceed f'_m, the specified compressive strength of masonry.

6.2.3 Specified Masonry Compressive Strength

The primary design parameter for masonry is f'_m, the specified compressive strength of masonry. This parameter is normally not critical for design because the flexural capacity of tension-controlled sections is insensitive to f'_m, and the shear capacity of masonry is proportional to the square root of f'_m. As a consequence, it is normally not cost-effective to specify higher values of f'_m than are customary for local construction. One exception to this is wall segments whose design is limited by maximum permissible percentages of vertical (longitudinal) reinforcement. Those maximum limits can be significantly increased by increasing f'_m. Additionally, if compliance with the specified f'_m is verified by prism testing rather than by the unit strength method, higher values of f'_m can often be justified with the same benefits.

6.2.4 Reinforcement

TMS 402 §9.1.9.3 requires that the specified yield strength of deformed reinforcement not exceed 60,000 psi, and §9.1.9.3.1 prohibits the use of deformed reinforcement whose yield strength exceeds 1.3 times the specified yield strength. The intent of these provisions is to require reinforcement with a significant yield plateau and to ensure that the probable flexural capacity (and consequent shear demand) in masonry elements does not exceed the $1.25M_n$ used in calculations for capacity design for shear.

6.3 Diaphragms

Diaphragm design is addressed in ASCE 7 §12.10, and further guidance is provided in the NEHRP Seismic Design Technical Briefs on *Cast-in-Place Concrete Diaphragms, Chords, and Collectors* (NIST 2010) and *Composite Steel Deck and Concrete-filled Diaphragms* (NIST 2011). In the particular context of masonry design, the designer should note that when shear-dominated masonry walls attract forces larger than those consistent with a response modification factor, R, equal to 5, diaphragms and their connections must resist those larger forces as well.

7. Detailing and Constructability Issues

Construction practices for masonry vary regionally in the United States, and the designer should be familiar with local practices to ensure that planned details can be built economically.

7.1 Selecting Masonry Units

Hollow concrete and hollow clay masonry units are available in a variety of shapes. For special walls, particularly those that are fully grouted with high percentages of vertical reinforcement, two special shapes of hollow masonry units can improve constructability. Both are more widely available on the west coast of the United States than elsewhere.

Single or double open-end units can be placed around vertical reinforcement that is already in place, which can be advantageous in reducing the number of lap splices.

Knock-out blocks have precut webs that can easily be removed with a hammer to allow placement of horizontal bars in the resulting trough. They also allow horizontal flow of grout between adjacent cells.

Figure 7-1 shows single open-end blocks that are also knock-out blocks. Double open-end units are shown in use in **Figure 7-2**. A vibrator in one cell can effectively consolidate grout in adjacent cells. Furthermore, the gap that normally occurs between adjacent end webs at head joints is solidly filled with grout, creating an uninterrupted shear plane.

Figure 7-2. Grout flow is facilitated in a fully grouted wall with open-end masonry units.

7.2 Details of Reinforcement

As discussed previously, uniformly distributed vertical reinforcement is usually preferable to over vertical reinforcement concentrated at wall ends. TMS 402 §9.3.3.1 encourages these detailing practices through limitations on the maximum diameters of reinforcing bars and on the percentages of reinforcement that can occupy a cell or a course of hollow-unit construction lap splices and on the maximum bar dimension relative to the grout space.

Shear reinforcement in special walls must be anchored around end vertical bars with a standard hook in a horizontal plane as per TMS 402 §7.3.2.6(f), as shown in **Figure 7-3**.

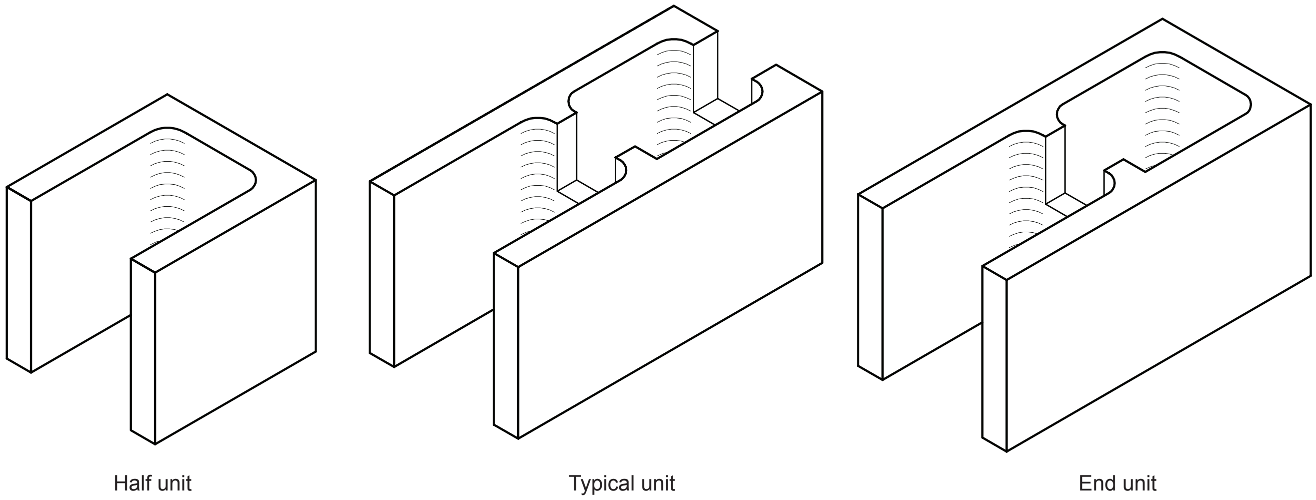


Figure 7-1. Special open-end shapes of hollow masonry units.

Figure 7-3. Anchorage of horizontal bars around vertical bars with a standard hook in a horizontal plane.

7.3 Joint Reinforcement

Joint reinforcement is often used in areas other than the Western United States as shear reinforcement for controlling the distribution and width of shrinkage cracks and also for attaching clay masonry veneer. Joint reinforcement is made of cold-drawn steel wires electrically welded together. It may be annealed but usually is not. It is normally galvanized and is also available in stainless steel. It is available in ladder configuration (with perpendicular cross wires) and truss configuration (continuous diagonal cross wires) conforming to ASTM A951 (ASTM 2014). The ladder configuration is preferable because the perpendicular cross wires offer less obstruction to grout and vibrators. This is particularly important for high-lift grouting.

Because of concerns about ductility (Shing and Noland 1992), earlier editions of TMS 402 prohibited the use of joint reinforcement to resist design shears while permitting its use to meet prescriptive requirements. More recent research (Baenziger and Porter 2010) has shown that joint reinforcement can participate effectively as shear reinforcement, provided that it meets requirements for minimum cross-sectional area. This finding is reflected in the 2013 edition of TMS 402 §9.3.3.7. The specified yield strength of the joint reinforcement used as shear reinforcement is typically 70,000 psi but it is limited to a maximum of 85,000 psi in accordance with TMS 402 §9.9.3.2.

7.4 Grouting

Reinforced masonry designed according to TMS 402 and constructed according to TMS 602 requires the proper specification, placement, consolidation, and reconsolidation of grout. Grout should be placed with a slump of 8 to 11 inches to ensure that it completely fills the spaces to be grouted. It should be consolidated to eliminate voids and reconsolidated to compensate for the loss of volume because of absorption of water by the surrounding masonry (**Figure 7-4**).

Figure 7-4. Consolidation of grout using a vibrator.

Design choices that can improve the grouting process include grouting admixtures, self-consolidating grout, using open-end and knock-out units, and reducing the number of bar splices or mechanical couplers.

8. References

ACI (2011). *Building code requirements for structural concrete* (*ACI-318-11*) *and commentary*, American Concrete Institute, Farmington Hills, MI.

Ahmadi, F. (2012). *Displacement-based seismic design and tools for reinforced masonry shear-wall structures*, Ph.D. dissertation, Department of Civil and Environmental Engineering, The University of Texas at Austin, TX.

Ahmadi, F., Mavros, M., Klingner, R. E., Shing, B., and McLean, D. (2014a). "Displacement-based seismic design for reinforced masonry shear-wall structures, part 1: Background and trial application," *Earthquake Spectra*, published online December 20, 2013; doi: http://dx.doi.org/10.1193/120212EQS344M.

Ahmadi, F., Mavros, M., Klingner, R. E., Shing, B., and McLean, D. (2014b). "Displacement-based seismic design for reinforced masonry shear wall structures, part 2: Validation with shake table tests," *Earthquake Spectra*, published online December 20, 2013; doi: http://dx.doi.org/10.1193/120212EQS345M.

ASCE (2010). *Minimum design loads for buildings and other structures* (*ASCE/SEI 7-10*), American Society of Civil Engineers, Reston, VA.

ASTM (2010). *Standard specification for grout for masonry*, ASTM C476. ASTM International, West Conshohocken, PA.

ASTM (2012). *Standard test method for compressive strength of masonry prisms*, ASTM C1314. ASTM International, West Conshohocken, PA.

ASTM (2014) *Standard specification for steel wire for masonry joint reinforcement*, ASTM A951/A951M. ASTM International, West Conshohocken, PA.

Baenziger, G., and Porter M. L. (2010). *In-plane structural testing of joint reinforcement in concrete masonry shear walls*, Department of Civil, Construction, and Environmental Engineering, Iowa State University, Ames, IA.

Banting, B. R., and El-Dakhakhni, W. W. (2012). "Force- and displacement-based seismic performance parameters for reinforced masonry structural walls with boundary elements," *Journal of Structural Engineering*, Vol. 138, No. 9, pp. 1477-1491.

BIA (2014). *The technical notes on brick construction*, Brick Industry Association, Reston, VA, accessed May 2014, http://www.gobrick.com/Technical-Notes.

Cyrier, W. B. (2012). *Performance of concrete masonry shear walls with integral confined concrete boundary elements*, master's thesis, Department of Civil and Environmental Engineering, Washington State University, Pullman, WA.

FEMA (1999). *Evaluation of earthquake-damaged concrete and masonry wall buildings: Basic procedure manual*, Federal Emergency Management Agency, Washington, D.C.

FEMA (2009). *NEHRP Recommended provisions for seismic regulations for new buildings and other structures*, FEMA P-750 2009 edition, Washington, D.C.

He, L. and Priestley, M. J. N. (1992). *Seismic behavior of flanged masonry shear walls,* Report No. 4.1-2, U.S.-Japan Coordinated Program for Masonry Building Research, Department of Structural Engineering, University of California, San Diego, La Jolla, CA.

IBC (2012). *International building code*, International Code Council, Washington, D.C.

Kingsley, G. R., Seible, F., Priestley, M. J. N., Hegemier, G.A. (1994). *The U.S.-TCCMAR full-scale five-story masonry research building test, part II: design, construction, and test results*, Structural Systems Research Project, Report No. SSRP-94/02, Department of Structural Engineering, University of California, San Diego, La Jolla, CA.

Klingner (2010). Klingner, R. E., *Masonry Structural Design*, McGraw-Hill Companies, Inc., New York, NY.

Lepage, A., Dill, S., Haapala, M. and Sanchez, R. (2011). "Seismic design of reinforced masonry walls: current methods and proposed limit-design alternative," *Proceedings*, *11th North American Masonry Conference*, Minneapolis, MN, The Masonry Society, Longmont, CO.

Lepage, A., and Sanchez R. (2012). "Practical nonlinear analysis for limit design of reinforced masonry walls," *The Open Civil Engineering Journal*, Vol. 6, pp. 107-118.

Merryman, K. M., Leiva, G., Antrobus, N., and Klingner, R. E. (1990). *In-plane seismic resistance of two-story concrete masonry coupled shear walls*, PMFSEL Report No. 89-3, Phil M. Ferguson Structural Engineering Laboratory, The University of Texas at Austin, TX.

Minaie, E., Mota, M., Moon, F. L., and Hamid, A. A. (2010). "In-plane behavior of partially grouted reinforced concrete masonry shear walls," *Journal of Structural Engineering*, Vol. 136, No. 9, pp. 1089-1097.

Morrison, S. C., and Bennett, R. M. (2013). "Concepts of shear friction applied to masonry shear walls." *The Masonry Society Journal*, Vol. 31, No. 1, pp. 1-13.

Murcia-Delso, J., and Shing, P. B. (2012). "Fragility analysis of reinforced masonry shear walls." *Earthquake Spectra*, Vol. 28, No. 4, pp. 1523-1547.

NCMA (2010). *Control joints for concrete masonry walls-empirical method*, TEK 10-2C, National Concrete Masonry Association, accessed July 10, 2014, http://www.ncma-br.org/e-tek.asp.

NIST (2010). *Seismic design of cast-in-place concrete diaphragms, chords, and collectors: A guide for practicing engineers*, NIST GCR 10-917-4, NEHRP Seismic Design Technical Brief No. 3, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD.

NIST (2011). *Seismic design of composite steel deck and concrete-filled diaphragms: A guide for practicing engineers*, NIST GCR 11-917-10, NEHRP Seismic Design Technical Brief No. 5, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD.

NIST (2012). *Seismic design of cast-in-place concrete special structural walls and coupling beams: A guide for practicing engineers*, NIST GCR 11-917-11REV-1, NEHRP Seismic Design Technical Brief No. 6, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD.

Noland, J. L., and Kingsley, G.R. (1995). "U.S. coordinated program for masonry building research: technology transfer," in *Research transformed into practice: Implementation of NSF research*, edited by Colville, J. and Amde, A., ASCE E-book accessible at http://ascelibrary.org/doi/book/10.1061/9780784400944, pp. 360-371, American Society of Civil Engineers, Reston, VA.

Paulay T., and Priestley, M. J. N. (1992). *Seismic design of reinforced concrete and masonry buildings*, John Wiley & Sons, Inc., New York, NY.

Priestley, M. J. N. and Elder, D. M. (1983). "Stress-strain curves for unconfined and confined concrete masonry," *ACI Journal*, Vol. 80, No. 3, pp. 192-201.

Seible, F., Kingsley, G. R., Priestley, M. J. N., and Kurkchubasche, A. G. (1991). *Flexural coupling of topped hollow core plank floor systems in shear wall structures*, Report No. SSRP 91/10, Department of Structural Engineering, University of California, San Diego, La Jolla, CA.

Sherman, J. D. (2011). *Effects of key parameters on the performance of concrete masonry shear walls under in-plane loading*, master's thesis, Department of Civil and Environmental Engineering, Washington State University, Pullman, WA.

Shing, P. B., Noland, J. L., Klamerus, E., and Spaeh, H. (1989). "Inelastic behavior of concrete masonry shear walls," *Journal of Structural Engineering*, Vol. 115, No. 9, pp. 2204-2225.

Shing, P. B., and Noland, J. L. (1992). *Shear behavior of concrete masonry walls with horizontal joint reinforcement*, research report, University of Colorado, Boulder, CO.

Shing, P. B., Carter, E. W., and Noland, J. L. (1993). "Influence of confining steel on flexural response of reinforced masonry shear walls," *The Masonry Society Journal*, Vol. 11, No. 2, pp. 72-85.

TMS (2013a). *Building code requirements for masonry structures* (*TMS 402/ACI 530/ASCE 5*), The Masonry Society, Longmont, CO.

TMS (2013b). *Specifications for masonry structures* (*TMS 602/ACI 530.1/ASCE 6*), The Masonry Society, Longmont, CO.

TMS (2013c). *Masonry designers' guide MDG-7*, The Masonry Society, Longmont, CO.

9. Notations and Abbreviations

A_g	gross cross-sectional area of a member, in^2
A_n	net cross-sectional area of a member, in^2
A_s	area of nonprestressed longitudinal tension reinforcement, in^2
c	distance from the fiber of maximum compressive strain to the neutral axis, inches
C_d	deflection amplification factor as given in ASCE 7
C^w	axial compression force induced in a wall by the action of coupling elements, lb
d	distance from extreme compression fiber to centroid of tension reinforcement, inches
d_v	actual depth of member in direction of shear considered, inches
E_m	modulus of elasticity of masonry in compression, psi
E_s	modulus of elasticity of steel, psi
f'_m	specified compressive strength of masonry, psi
f_y	specified yield strength of steel reinforcement, psi
$I_{e,}$	effective moment of inertia, in^4
$I_{g,}$	moment of inertia of gross cross-sectional area of a member, in^4
M	maximum moment in a wall element, in-lb
M_n	nominal moment strength, in-lb
M_u	factored moment, in-lb
$M_{u,0}$	factored out-of-plane moment, in-lb
M_{pi}^b	plastic moment capacity of coupling element at level *i*, in-lb
M_{pi}^w	plastic moment capacity of wall element *i*, in-lb
$M_u/(V_u d_v)$	shear-span-to-depth ratio for SD
$M/(Vd_v)$	shear-span-to-depth ratio for ASD
P	axial load, lb
P_u	factored axial load, lb
R	response modification coefficient
T^w	axial tension force induced in a wall by the action of coupling elements, lb
V	shear force in a wall element, lb
$V_{flexure}$	shear corresponding to the flexural capacity of wall element, lb
V_{lim}	limiting base shear strength, lb
V_M^b	shear corresponding to the plastic moment capacity of coupling element, lb
V_n	nominal shear strength, lb
V_{nf}	nominal shear-friction strength, lb
V_{nm}	nominal shear strength, provided by masonry, lb
V_{ns}	nominal shear strength provided by shear reinforcement, lb
V_{shear}	shear corresponding to the shear capacity of wall element, lb
V_u	factored shear force, lb
V_i^w	shear corresponding to the plastic moment capacity of wall element *i*, lb
α	tensile strain factor used in the calculation of the maximum permissible area of flexural tensile reinforcement
ρ	redundancy factor defined in ASCE 7
ρ_h	horizontal reinforcement ratio
ρ_v	vertical reinforcement ratio
ρ_{max}	maximum flexural tension reinforcement ratio
$\rho_{req'd}$	reinforcement ratio required by analysis
μ	coefficient of friction
μ_Δ	displacement ductility
μ_ϕ	curvature ductility

ϕ	strength reduction factor
Δ	displacement, inches
γ_f	reduction factor used in shear-friction calculation
χ	ratio of the area of vertical reinforcement in compression to the total area of vertical reinforcement crossing the shear plane, used in shear-friction calculation

Abbreviations

ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ASD	allowable stress design
ASTM	ASTM International, previously known as the American Society for Testing and Materials
ATC	Applied Technology Council
BSSC	Building Seismic Safety Council
CUREE	Consortium of Universities for Research in Earthquake Engineering
FEMA	Federal Emergency Management Agency
IBC	International Building Code
MSJC	The Masonry Standards Joint Committee
NCMA	National Concrete Masonry Association
NEHRP	National Earthquake Hazards Reduction Program
NIST	National Institute of Standards and Technology
SD	strength design
SEI	Structural Engineering Institute of the American Society of Civil Engineers
SFRS	seismic force-resisting system
TCCMAR	Technical Coordinating Committee for Masonry Research
TMS	The Masonry Society

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