Design of special reinforced masonry shear walls in the U.S.

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ABSTRACT: The paper summarizes a new, practical guide titled "Seismic Design of Special Reinforced Masonry Shear Walls, A Guide for Practicing Engineers" that is available for free online at http://www.nehrp.gov/library/techbriefs.htm. While focused on the U.S. codes for masonry design, the Guide addresses universal challenges relevant to masonry seismic design in any country. Masonry serves simultaneously as architecture (defining the building's appearance and functional program), enclosure (defining the building's external envelope), and structure (resisting vertical and lateral loads). The architectural elements are established first; consequently, the structural designer is not able to choose the location and proportion of masonry structural elements, but must be able to detail the given elements to meet the intent of the code for stiffness, strength, and ductility. The Guide provides insight regarding analysis and design for different wall configurations, application of rules and limitations on reinforcement, and application of newly codified limit design procedures.

1 INTRODUCTION

In 2014, a practical guide titled "Seismic Design of Special Reinforced Masonry Shear Walls, A Guide for Practicing Engineers" (referred to in the following as "The Guide") was published by the National Institute of Standards and Technology (NIST) in the U.S. It is relevant not only for guiding the designer, but for identifying important issues that may not (yet) be addressed explicitly in the code itself.

The Guide focuses on the design of Special Reinforced Masonry Shear Walls subjected to in-plane seismic and gravity loads. In spite of the intention of the code provisions, these walls are distinguished by two fundamental types of behavior:

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

The Guide recognizes this fundamental challenge unique to masonry design: other construction materials 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, in 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 does not get 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 given elements that configure the space, and be able to anticipate the expected behavior of those elements so that they may be designed and detailed appropriately. This is true for structural walls in all seismic design categories, but can be particularly challenging for special walls, because the expected level of ductility implied by the "special" designation simply may not be available.

2 WHAT ARE "SPECIAL" WALLS?

The *International Building Code* (IBC 2012) requires the use of special reinforced masonry walls whenever masonry structural walls are used to resist seismic forces in new buildings assigned to Seismic Design Category D, E, or F. These design force levels are specified in *Minimum Design Loads for Buildings and Other Structures* (ASCE 2010), and the design procedures and detailing requirements are addressed in the 2013 edition of *Building Code Requirements for Masonry Structures* (TMS 2013), referred to in the following as "ASCE 7" and "TMS 402" respectively. In ASCE 7, special walls are assigned the highest response modification factor, *R*, of any of the masonry shear wall types. For bearing wall systems $R = 5$; for building frame systems, $R = 5.5$. Inherent in the use of an *R* factor of 5 or more is the presumption of ductile behavior, associated with the development of plastic hinges with stable inelastic rotation capacity. The particular challenge of masonry seismic design addressed in the Guide is that the designer cannot presume that following the prescriptive requirements of TMS 402 will necessarily ensure the ductile, flexure-dominated behavior assumed in the determination of the design seismic loads.

3 DESIGN PRINCIPLES FOR SPECIAL REINFORCED MASONRY SHEAR WALLS

3.1 *Shear Wall Configurations in Buildings*

Masonry shear walls can have a variety of plan and elevation configurations. Several common configurations are shown in Figure 1. Ultimately, the behavior of these different configurations is related to the collective behavior of multiple wall elements, each with its own aspect ratio, axial load, and reinforcement. These issues are discussed in detail in the Guide.

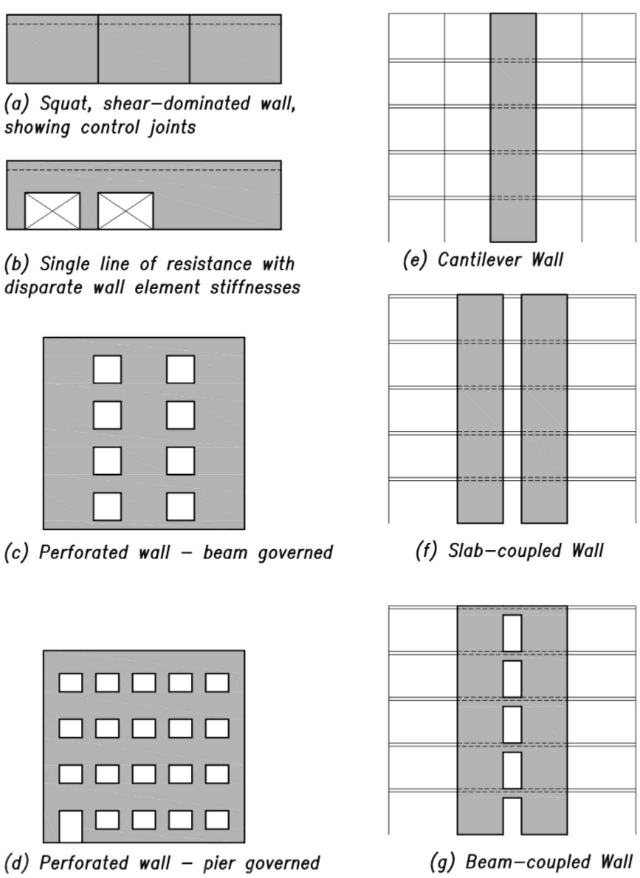

Figure 1. Common wall configurations.

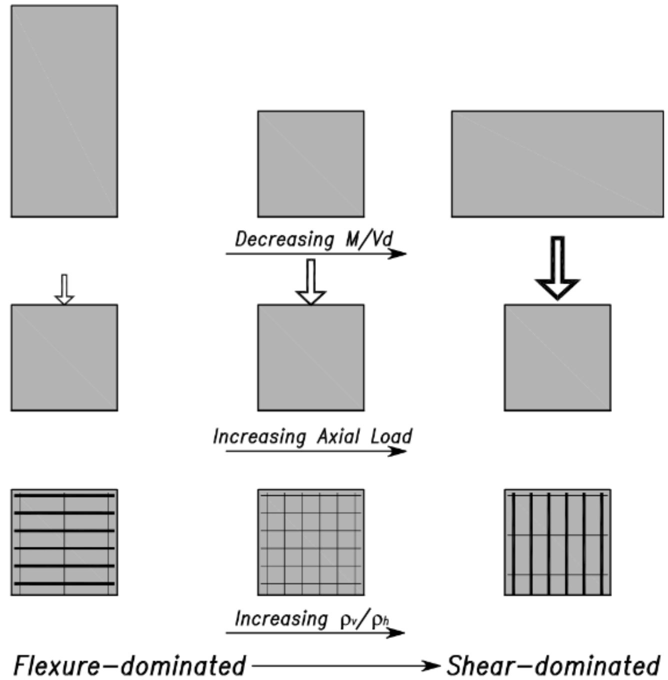

Figure 2. Conceptual illustration of the influence of shear-span ratio, axial load, and ratio of vertical to horizontal reinforcement on wall behavior.

3.2 *Flexure- versus Shear-dominated Walls*

Flexure-dominated walls are generally ductile, with behavior characterized by extensive yielding of vertical reinforcement, and ultimately, localized crushing at the compression toe of the wall. Shear-dominated walls are generally brittle, with failure characterized by diagonal shear cracks. Examples of each from laboratory tests are shown in the Guide. The implicit goal of TMS 402 is that special masonry shear walls be flexure-dominated walls. The code indirectly encourages designs that meet this goal through capacity based design for shear, maximum reinforcement restrictions, and prescriptive requirements, such as distribution of reinforcement and limitations on bar diameters, and other provisions.

Figure 2 illustrates the factors that may lead to shear dominated behavior in a qualitative way. The Guide provides a more quantitative illustration of the influence of aspect ratio, axial load, and ratio of vertical to horizontal reinforcement on behavior.

3.3 *Maximum Vertical Reinforcement Requirements*

The requirements of TMS 402 §9.3.3.5 for strength design 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.

It can be challenging to meet design requirements that limit the total amount of vertical reinforcement in special walls. The maximum permissible reinforcement percentage is much less for special walls than for ordinary walls, and it decreases further as the design axial force increases.

Maximum reinforcement requirements do not apply in some cases, such as for squat walls with $M_u/(V_u d_v) < 1.0$, or when certain provisions associated with boundary element reinforcement are satisfied.

3.4 *History of the Development of Maximum Vertical Reinforcement Requirements*

It is worthwhile to understand the basis of the maximum reinforcement requirements from an historical perspective. It has been recognized for quite some time that, in order to obtain the ductile behavior that is consistent with code objectives for reinforced masonry walls subjected to seismic loading, the yielding of flexural reinforcement prior to masonry crushing at the extreme compression fiber is the desired failure mode. This cannot be achieved in over-reinforced masonry shear walls in which a significant portion of the flexural reinforcement may not yield prior to the masonry compression zone reaching the maximum usable strain. Hence, limiting flexural reinforcement is an appropriate strategy for the design of flexure dominated masonry shear walls subjected to seismic loading.

The original basis and rationale for the current maximum reinforcement requirements for special reinforced masonry shear walls in Section 9.3.3.5.3 of TMS 402-13 (TMS 2013) can be found in the 2000 NEHRP Provisions and Commentary (NEHRP 2001a, b). The commentary states,

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

The referenced table includes the same allowable story drift limits specified in Table 12.2-1 of ASCE 7-10 - Minimum Design Loads for Buildings and Other Structures (ASCE 2010).

The NEHRP Provisions required the designer to assume a strain profile across the flexural section of the wall. On the compression side of the profile, the maximum usable masonry strain was limited to 0.0025 for concrete masonry or 0.0035 for clay masonry. On the tension side of the profile, the reinforcing bar furthest away from the edge of the compression zone was required to deform an amount equal to five times the yield strain of that bar.

The intent of the maximum reinforcing limit requirement is to ensure that the majority of the in-plane flexural reinforcement has been fully engaged and yielded in order to develop the maximum possible moment capacity of the wall by the time the wall is displaced to or past the drift limit state, and before the masonry strain in the compression zone of the wall reaches the designated maximum usable strain, which is assumed to be the onset of toe crushing. For uniformly distributed vertical wall reinforcement, this requires that the reinforcing bars furthest from the edge of the compression zone experience strains well past the yield strain. This criterion not only limits toe crushing, but prevents significant additional compression from developing in the masonry once toe crushing has occurred, thus eliminating the need for confined boundary elements used in reinforced concrete shear walls.

The requirement to develop five times the yield strain was based upon the relationship between the inelastic curvature demand in the plastic hinge region of a cantilever wall and the target displacement (drift) at the top of the wall, (Paulay & Priestley 1992). With reference to Figure 3, the following equations describe that relationship:

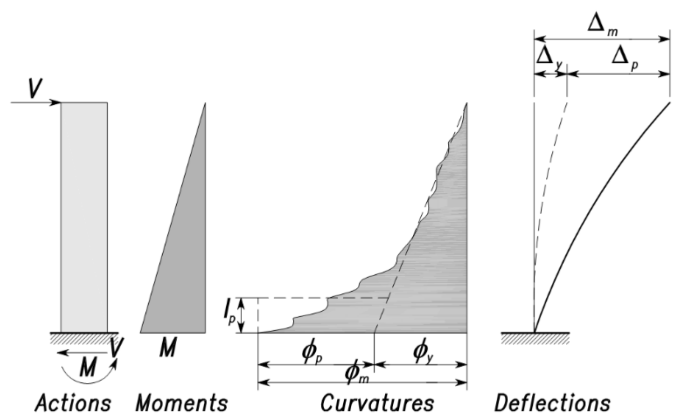

Figure 3. Moment, curvature, and deflection relationships for a prismatic reinforced masonry cantilever wall.

$$\Delta_y = \frac{\phi_y \lambda^2}{3} \tag{1}$$

$$\theta_p = \phi_p \lambda_p = (\phi_m - \phi_y)\lambda_p \tag{2}$$

$$\Delta_p = \theta_p(\lambda - 0.5\lambda_p) = \phi_p \lambda_p (\lambda - 0.5\lambda_p) \tag{3}$$

$$\Delta_m = \Delta_y + \Delta_p \tag{4}$$

where

Δ_p = plastic displacement at top of wall
Δ_y = yield displacement at top of wall
Δ_m = total displacement at the top of the wall
λ_p = length of plastic hinge
θ_p = plastic rotation occurring in the plastic hinge
ϕ_y = yield curvature
ϕ_p = plastic curvature occurring at the plastic hinge location

ϕ_m = total curvature occurring at the plastic hinge location
λ = height of shear wall
d = length of the shear wall
ε_s = steel strain in the extreme tensile bar

If we assume that elastic deformation is small compared to inelastic deformation, then: Δ_y and ϕ_y = 0, and $\phi_p = \phi_m$. Then equation (4) becomes $\Delta_m = \Delta_p$.

At the maximum allowable drift ratio specified in the Provisions, $\Delta_p = 0.01\lambda$. Thus,

$$0.01\lambda = \phi_p \lambda_p (\lambda - 0.5\lambda_p) \quad (5)$$

With a maximum usable strain for concrete masonry of 0.0025, we can express the curvature at the plastic hinge location as:

$$\phi_p = \frac{(0.0025 + \varepsilon_s)}{d} \quad (6)$$

Substituting this value for ϕ_p in equation (5) yields:

$$\frac{0.01\lambda}{\lambda_p(\lambda - 0.5\lambda_p)} = \frac{(0.0025 + \varepsilon_s)}{d} \quad (7)$$

or:

$$\varepsilon_s = \left[\frac{0.01\lambda}{\lambda_p(\lambda - 0.5\lambda_p)}\right] d - 0.0025 \quad (8)$$

With the assumption stated in the NEHRP Commentary that λ_p = d and $\lambda - 0.5\lambda_p \approx \lambda$, equation (8) yields $\varepsilon_s = 0.0075$. For ASTM 615 grade 60 reinforcing bars, the yield strain is 0.00207, and 0.0075 is 3.6 times the yield strain. However, as explained in the 2000 NEHRP commentary, "because the inelastic rotation is not concentrated at a single point, and because the inelastic hinge length can be less than the plan length of the wall", the required minimum tensile strain was set to a more conservative value of 5 times the yield strain. In addition, the designer should note that the degradation of the masonry compression zone can result in a rapid vertical splitting failure, which is different than the behavior of concrete shear wall compression zones, because of the lack of confinement.

The NEHRP limitation on the vertical reinforcement has the effect of ensuring that 80% of the tensile steel reaches a strain level equal to or greater than yield prior to toe crushing. This recommendation was adopted in the 2000 International Building Code (International Code Council 2000).

As shown in equation (8), the value of ε_s is sensitive to the assumption made for the length of the plastic hinge. The NEHRP commentary assumption is that the length of the plastic hinge is equal to the plan length of the shear wall. Pauley and Priestley (Paulay & Priestley 1992) state that for typical shear wall proportions, the length of the plastic hinge is likely to be half that, or 0.5d. A more recent text (Priestley et al. 2007) recommends a plastic hinge length based upon the following equation:

$$\lambda_p = \left[(0.04\lambda) + (0.1d) + (0.15 d_b f_y)\right] \geq 3(0.15 d_b f_y) \quad (9)$$

where d_b (inches) is the diameter of the vertical reinforcing in the wall at the hinge and f_y (ksi) is the specified yield strength of the vertical reinforcement. This equation increases the plastic hinge length slightly by taking into account the additional length of strain penetration of the vertical wall reinforcing bars embedded in the foundation.

Clearly, the value of ε_s determined using equation (8) is only as accurate as the assumption for the plastic hinge length. The assumption that the plastic hinge length is equal to the wall length is only valid for walls with certain aspect ratios (λ/d). Also, the required minimum tensile strain does not account for configuration differences such as "I-shaped" or "L-shaped" walls. These issues were not specifically acknowledged in the original NEHRP provisions or the 2000 IBC.

The plastic hinge length assumed in the 2000 NEHRP provisions is not accurate for shear walls with small aspect ratios. For these types of walls, the plastic hinge length will be much shorter than the wall length. If this is taken into account, the required steel strain will exceed 5 times the yield at the drift limit of 0.01, according to equation (8). In this case, having $\varepsilon_s = 0.0075$ will not result in the ductility that was anticipated in the provisions. Hence, the NHERP provisions as adopted in TMS 402 will not ensure uniform flexural ductility for walls of different aspect ratios.

Furthermore, the use of the maximum allowable drift ratio of 0.01 to compute ε_s introduces a fundamental issue for long squat walls. It is very unlikely that long squat walls will ever reach the code specified drift limit because of the high stiffness to the load demand ratio. These walls deform primarily in shear and generally behave as shear dominated elements. However, since the moment capacity of a wall is proportional to the square of the length of the wall cross section if the steel ratio remains the same, long squat walls are unlikely to be over-reinforced. In recognition of this, a provision was added in the 2005 Building code Requirements for Masonry Structures (TMS 2005) to exclude long squat walls from the maximum reinforcement ratio. It states, "For masonry members where $M_u/V_u d_v \leq 1$ and when designed using $R \leq 1.5$, there is no upper limit to the maximum flexural tensile reinforcement."

A moment-curvature analysis study was done to evaluate the basis and validity of the 2000 NHERP Provisions and 2000 IBC maximum reinforcement requirements (Tallon et al, 2002). The results of the study confirmed that an upper limit on reinforcement was not required for short squat walls. In addition, the study recommended that the influence of the aspect ratio of the wall should be considered when determining the required curvature. It was expressed that the 2000 NHERP Provisions and 2000 IBC maximum reinforcement requirements were overly conservative for shear walls with aspect ratios greater than 2.5.

In the 2005 Building Code Requirements for Masonry Structures (TMS 2005), the strain criteria was revised from 5 to 4 for special masonry shear walls. The rationale for the change, as stated in the commentary to the Code, was based on the assumption that that the target curvature ductility can be associated directly with the target displacement ductility as represented by the C_d factor. The C_d factor for special reinforced masonry shear walls is given in ASCE 7: for bearing wall systems, C_d is 3.5, and for building frame systems, C_d is 4.0.

3.5 *Perforated Walls and the Limit Design Method*

Perforated walls (Figure 1b, 1c, and 1d) can be particularly challenging because wall elements between openings normally have low shear-span to depth ratios, and may have high axial loads as well; they are therefore vulnerable to shear-dominated behavior. Appendix C (Limit Design) of TMS 402 provides an alternative way of designing special walls that is particularly beneficial for perforated walls, and addresses behavior modes explicitly.

Limit Design can be applied to individual lines of resistance in structures that are otherwise designed according to the strength design requirements of Chapter 9. It allows the structural designer to explicitly take into account the anticipated plastic mechanism of the wall system, to control the aspect ratios and reinforcement of wall elements to achieve the best behavior possible, and to detail the elements in accordance with the resulting flexure- or shear-dominated behavior. To determine the required design strengths of each wall segment, Limit Design requires plastic limit analysis, which is also discussed in the Guide.

4 DESIGN GUIDANCE

The Guide includes a flow chart that provides a step by step presentation of the design process. Ultimately, the designer is encouraged to take additional steps – beyond code requirements – to establish whether the special wall is flexure-dominated (the implicit code intent for special walls) or shear-dominated (the unfortunate but unavoidable result of some architectural wall configurations). When a wall is shear-dominated, options are presented to achieve flexure dominated behavior, or, if that is not possible, to design for shear dominated behavior in a responsible way using a capacity design approach, and recognizing the reduced ductility of these wall elements. The designer is encourage to recognize that when shear-dominated masonry walls are designed with the understanding that they will attract forces larger than those consistent with a response modification factor, *R*, diaphragms and their connections must resist those larger forces as well.

5 CONCLUSION

It is hoped that practitioners who read "Seismic Design of Special Reinforced Masonry Shear Walls, A Guide for Practicing Engineers" will find not only a concise guide to current practice for special walls, but also a glimpse at the currently evolving direction of codes for seismic design of masonry. Reinforced masonry has unique challenges, and this short guide should make those challenges a bit more understandable for all.

As a brief introduction to the Guide, this paper has addressed a few of the issues relevant to design of special walls:

- Special walls designed in accordance with the code are intended to exhibit ductile, flexural behavior, but forces beyond the designer's control can result in shear dominated behavior rather than the expected flexure dominated behavior, thus affecting the design forces for the wall and adjacent elements.
- Code imposed limitations on vertical, flexural reinforcement are intended to ensure ductile behavior, but can be challenging to meet. A brief theoretical and historical basis of the provisions is presented here.
- Perforated walls are particularly susceptible to shear dominated behavior.
- New Limit Design provisions in an appendix to TMS 402 provide options for designers faced with some of the particular challenges of special reinforced masonry walls.

6 ACKNOWLEDGEMENTS

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7 DEDICATION

This paper is respectfully dedicated to the memory of Professor M.J.N. Priestley whose visionary work over many decades provided much of the foundation for the design principles discussed in the Guide.

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